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LONGMANS' CIVIL ENGINEERING SERIES

NOTES ON CONSTRUCTION
IN MILD STEEL

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NOTES
ON
CONSTRUCTION IN MILD STEEL

ARRANGED FOR THE USE OF JUNIOR DRAUGHTSMEN IN THE ARCHITECTURAL AND ENGINEERING PROFESSIONS

WITH ILLUSTRATIONS FROM WORKING DRAWINGS, DIAGRAMS, AND TABLES

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PREFACE

In the collection of "Notes" on Mild Steel and Constructional Steelwork, which form the contents of the present volume, no attempt has been made to treat the subject from the point of view of Applied Mechanics as ordinarily understood, nor are the theories of construction, nor the calculations for buildings or engineering structures referred to, except so far as may be required incidentally in connection with the subject-matter discussed, while the great range of the subjects upon which the Notes treat, and the severe limitations which are necessarily imposed, must form the only excuse which the writer has to offer for the obvious insufficiency of treatment of the items dealt with.

It has been assumed that the junior draughtsman of the architectural or engineering professions has been, at least to some extent, properly grounded in the theory of construction, and that he has acquired an elementary knowledge of the determination of stresses in the structures with which he has to deal.

It has, however, come within the experience of the writer that between the carefully calculated stress-sheet or correctly drawn graphic diagram, and the completion of a working drawing which shall successfully pass the ordeal of criticism in the girdermaker or bridge- or roof-builder's yard, there is sometimes found a gap, not always successfully bridged, and it becomes occasionally evident that the ability to produce, let us say, a correct graphic analysis of the stresses on a roof principal and the ability to design a sound riveted connection are not quite one and the same thing.

It is true that excellence and soundness of design are not to be acquired from books alone, and that close study, observation, and

experience must go hand-in-hand to arrive at that result. It is, however, the hope of the writer that the Notes now offered will assist the student, at all events, in the study and observation of such good examples of Steel Construction as may come within his reach, and in the practical application of a material which has taken, and is likely to maintain, so important a position in both Architectural and Engineering Construction.

The education of the designer of Constructional Steelwork is not, however, completed even when to a sound knowledge of theory he has added to that knowledge the experience of the practical aspects of design. He will, if he be wise, endeavour, so far as opportunity may be given him, to trace back the previous history of the material he has been dealing with; he will place himself, mentally and (as far as is possible to him) by personal observation, in touch with the centres of the Steel-making Industry, the Blast Furnace, the Cinder Heap, the "Sow" and her "Pigs," the dazzling radiance of the molten metal in the open hearth or the converter, the methods (to say nothing of the risks and anxieties) of the Steel Founder, the ruddy atmosphere of the Annealing Furnace, and the spectral shapes of castings, refracted by the waving and glowing gases as they undergo the ordeal which relieves internal stress and makes them ductile and tough, the Ingot, the Soaking Pit, the clangour, and hiss and roar of the Rolling Mills.

The scene changes, and he will follow the completed sections and shapes to the Plater's yard, the Templet-maker's, Machine and Smith's shops, the Pickling or Galvanizing Tanks, and watch the processes whereby Drilling Machine, Punching Machine, Riveting Machine, Pneumatic Hammer with its incessant rattle, Cold Saw with its halo of sparks, Hydraulic Press, and the like, shape and fashion his material into the form he has evolved on paper; and perhaps he then becomes conscious, as the offspring of his thought grows into visible bodily shape before his eyes, that there are certain details in his design which he will take care to improve on a future occasion.

Again the scene changes, the riveted sections of steelwork, the

castings, the cases of bolts and nuts, the bags of rivets, have all left the contractor's yard, some by rail, some perchance by sea, and then the multitudinous practical requirements which surround the Erection of Constructional Steelwork become evident, whether the Structure be some Bridge of great span over a ravine or rapid river, a Skeleton Steel "Skyscraper" many stories high, a large Caisson or Lock Gate for a Dock Entrance, or whether it be the simpler and humbler forms of Builders' Ironwork, and the erection of a few simple columns, girders, or roof principals.

All these things, and many more, noted with the observant eye, and the receptive and willing mind, will form so many rungs in the ladder whereby the junior draughtsman, be he Architect or Engineer, may climb, as regards this branch of his profession, to efficiency, success, and the honourable reward of his industry.

The application of Steel to that mode of construction known as "Ferro-Concrete," "Armoured Concrete," or "Concrete Steel" demands separate treatment, and is not alluded to in this volume. This subject, together with that of the protection of Constructional Steelwork from the effects of fire, the present writer must leave until such period as time and opportunity may indicate.

H. F.

LONDON,
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IT is the primary object of these notes to treat, as regards riveted work, of those combinations or assemblages of various rolled sections of steel which make up the constructional forms of steelwork to be dwelt upon in the pages following.

The methods by which these sections, amongst which may be enumerated plates, bars, angles, tees, joists, channels, and the like,

are produced in the rolling mills from the original ingot is beyond the scope of this work. It is, however, very desirable that the student and designer of structural work should possess some elementary knowledge at least of the leading features of the chemical and physical qualities of his material as affected by the various processes of manufacture, especially as these last are frequently referred to in modern specifications for structural steel-work, and the designer may be called upon to select that process which he considers best suited for his purpose, unless, indeed, he adopts the somewhat undesirable course of ignoring all reference to methods of manufacture, and is content to accept what is offered without further inquiry.

The following elementary and necessarily imperfect outline of these subjects has been therefore prepared, rather as an incentive to the student to prosecute further inquiry than as an attempt to treat even partially of a branch of metallurgy full of detail of absorbing interest.

Before treating of the several processes of manufacture now in vogue, it is desirable to consider briefly some of those chemical constituents which go to make up that compound of iron and carbon which is denominated steel under its various sub-divisions of hard, medium, soft, or mild.

The discussions which arose in the earlier days of modern steel manufacture as to the precise nomenclature of the various grades of steel, and especially as regards that quality of the metal now used for ordinary riveted structural work, have to-day less interest.

The appellation of "mild steel" is perfectly well understood, and the limits of the chemical and physical qualities of this metal, while they are subject to a certain amount of latitude with regard to the precise purposes in view, are nevertheless practically settled by the general consent of engineers and manufacturers.

The wonderful influence of a small percentage of carbon in combination with iron will at once attract the attention of the student, and the question of carbon content must now be entered upon.

The tabular statement which follows is intended to show the gradual increase in percentage of carbon which accompanies the increasing hardness of steel ranging from the softest quality manufactured, and applicable to those purposes which require great ductility, malleability, and welding properties, to those grades of steel standing at the summit of the scale of hardness, and used only for cutting instruments of the finest temper and edge.

The student will observe that the entire range or scale of carbon content is but about 1½ per cent.

It is unnecessary, perhaps, to point out that there is no hard-and-fast boundary line to be drawn between the several groups or strata of steels. Any one group may be found to overlap its neighbour to some slight extent, but in the main the percentages here given indicate within narrow limits those which will be found in chemical analyses of the metal used for the various purposes described.

TABLE No. 1.

SHOWING THE APPROXIMATE PERCENTAGES OF CARBON AND APPROXIMATE ULTIMATE TENSILE STRENGTH OF STEEL USED FOR THE PURPOSES DESCRIBED.

Class of material.	Percentage of carbon.	Approximate ultimate tensile strength in tons per sq. in.
Extra soft steel for such purposes as boiler flues or plates exposed to flame, rivets, tin-plates, tubes for boilers, welding material and the like	0·06–0·125	22–26
Mild steel for ship-building, bridge-work, builders' girders, riveted columns, roof trusses, rolled joists, trough floor sections, and the like	0·125–0·250	26–32
Medium steel for tyres, axles, rails for permanent way, railway vehicle springs, and the like	0·30–0·55	35–45
High Carbon Steels.		
Various blacksmiths' tools, and as weld steel for steeling[1]	0·65	Up to about 60 tons per square inch.
Wood-working chisel steel[1] ...	0·875	
Paving-tool steel, screw taps, chisels, gouges, etc.[1]	1·000	
Stocks and dies, draw-plates, etc.[1] ...	1·125	
Turning-tool steel, rock drills, mill picks, scrapers, and cutting tools for hard metals[1]	1·250	
Hard file steel[1]	1·375	
Razor steel, turning and planing knives, drills, turning gravers for very hard materials[1]	1·500	

[1] Skelton, "Economics of Iron and Steel." Also Thallner, "Tool Steel."

The above table is intended to exhibit broadly the relationship between carbon content and ultimate tensile resistance, but this relationship is not solely of this simple nature. The influence of the other chemical elements usually found in chemical analysis of mild steel, as affecting the practical working qualities and physical characteristics of the metal, must also be traced.

The following elementary remarks upon this subject are mainly based upon the work of a well-known American authority; and, except where otherwise mentioned, the passages in italics which follow are taken from the work referred to.[1]

Influence of Carbon.—"The ordinary steel of commerce is carbon-steel; in other words, the distinctive features of two different grades are due for the most part to variations in carbon rather than to differences in other elements.

"There are often wide variations in manganese, phosphorus, silicon, etc., but it is rarely that the carbon content does not determine the class to which the material belongs.

"This selection of carbon as the one important variable arose primarily from the fact that primitive Tubal Cains could produce a hard-cutting instrument with no apparatus save a wrought-iron bar and a pile of charcoal; and the natural developments in manufacture have led to the conclusion that a given content of carbon will confer greater hardness and strength, with less accompanying brittleness than any other element.

"There are certain exceptions to be taken to this statement in the case of hard steels made by manganese, chromium, or tungsten, but it may be accepted as true in soft steel.

"It follows, therefore, that no limit should ever be placed to the carbon allowed in any structural material if a given tensile strength is specified. It is, of course, true that every increment of carbon increases the hardness, the brittleness under shock, and the susceptibility to crack under sudden cooling and heating, while it reduces the elongation and reduction of area; but the strength must be bought at a certain cost, and this cost is less in the case of carbon than with any other element."

Influence of Silicon.—"The contradictory testimony concerning the effect of silicon on steel has been well summarized by Mr. Howe,[2] who records many examples of exceptional steels with

[1] Campbell, "Manufacture and Properties of Structural Steel."
[2] "The Metallurgy of Steel."

abnormal contents of silicon, and who fully discusses the theories advanced by different writers.

"He finds no proof that silicon has any bad effect upon the ductility or toughness of steel, and he concludes that the bad quality of certain specimens is not necessarily due to the silicon content, but to other unknown conditions."

In discussing the results of the investigation of Mr. Hadfield,[1] the following remarks are made by Mr. Campbell: "These results are of the highest value in showing that silicon cannot be classed among the highly injurious elements, for in similar proportion (the percentages of silicon in the investigations in question range from 0·21 to 5·08) phosphorus and sulphur would be out of the question, manganese would give a worthless metal, and carbon would change the bar to pig-iron. It will, therefore, be only reasonable to suppose that small quantities cannot exert a very deleterious influence."

Finally, the same author remarks that "in steels containing less than 0·25 per cent. of carbon, the effect of small proportions of silicon upon the ultimate strength is inappreciable."

Influence of Phosphorus.—"Of all the elements that are commonly found in steel, phosphorus stands pre-eminent as the most undesirable. It is objectionable in the rolling mill, for it tends to produce coarse crystallization, and hence lowers the temperature to which it is safe to heat the steel, and for this reason phosphoritic metal should be finished at a lower temperature than pure steel, in order to prevent the formation of a crystalline structure during the cooling."

"Aside from these considerations, its influence is not felt in a marked degree in the rolling mill, for it has no disastrous effect upon the toughness of red-hot metal when the content does not exceed 0·15 per cent."

"The action of phosphorus upon the finished material may not be dismissed in so few words. Mr. Howe[2] has gathered together the observations of different investigators, and the evidence seems to prove that the tensile strength is increased by each increment of phosphorus up to a content of 0·12 per cent., but that beyond this point the metal is weakened. Whether this last observation be correct or not is of little practical importance, for it would be criminal to use a metal for structural purposes that contained as much as an average of 0·12 per cent. phosphorus."

[1] "Alloys of Iron and Silicon." *Journal I. and S. I.*, vol. ii., 1889.
[2] "The Metallurgy of Steel."

"Below this point it is absolutely certain that phosphorus strengthens low steels, both acid and basic. . . . The same certainty does not pertain to any other effect of this metalloid. Mr. Howe has ably discussed the whole matter, and I herewith make quotations from the Metallurgy of Steel, and place them in the form of a summary.

"(1) The effect of phosphorus on the elastic ratio, as on elongation and contraction, is very capricious.

"(2) Phosphoric steels are liable to break under very slight tensile stress if suddenly or vibratorily applied.

"(3) Phosphorus diminishes the ductility of steel under a gradually applied load as measured by its elongation, contraction, and elastic ratio when ruptured in an ordinary testing machine, but it diminishes its toughness under shock to a still greater degree, and this it is that unfits phosphoric steels for most purposes.

"(4) The effect of phosphorus on static ductility appears to be very capricious, for we find many cases of highly phosphoric steel which show excellent elongation, contraction, and even fair elastic ratio, while side by side with them are others produced under apparently identical conditions but statically brittle.

"(5) If any relation between composition and physical properties is established by experience, it is that of phosphorus in making steel brittle under shock; and it appears reasonably certain, though exact data sufficing to demonstrate it are not at hand, that phosphoric steels are liable to be very brittle under shock, even though they may be tolerably ductile statically.

"The effects of phosphorus on shock-resisting power, though probably more constant than its effects on static ductility, are still decidedly capricious. . . ."

"It is true that numerous cases can be cited of rails, plates, etc., containing from 0·10 to 0·35 per cent. of phosphorus, which have withstood a long lifetime of wear and adversity; but in the general use of such metal there has been such a large percentage of mysterious breakages that it seems quite well proven that the phosphorus and the mystery are the same."

On the subject of phosphorus, another authority[1] remarks as follows:—

"In the case of what may be called the treacherousness of phosphoric steel, it is difficult to fix a definite limit for the

[1] F. W. Harbord, "The Metallurgy of Steel," 1904.

maximum content of phosphorus which can be safely allowed, but there can be no doubt that the lower this is, the safer the material, and for structural purposes 0·06 per cent. is quite as much as can be accepted with a feeling of security. In steel rails 0·08 per cent. of phosphorus may be permitted with safety."

Influence of Sulphur.—"Nothing is better established than the fact that sulphur injures the rolling qualities of steel, causing it to crack and tear, and lessening its capacity to weld. This tendency can be overcome in some measure by the use of manganese and by care in heating, but this does not in the least disprove that the sulphur is at work, but simply shows that it is overpowered.

"The critical content at which the metal ceases to be malleable and weldable varies with every steel. It is lower with each associated increment of copper, it is higher with each unit of manganese, and it is lower in steel which has been cast too hot.

"In the making of common steel for simple shapes, a content of 0·10 per cent. is possible, and may even be exceeded if great care be taken in the heating; but for rails and other shapes having thin flanges, it is advantageous to have less than 0·08 per cent., while every decrease below this point is seen in a reduced number of defective bars.

"It is impossible to pick out two steels with different contents of sulphur and say that the influence of a certain minute quantity can be detected, but it is none the less true that the effect of an increase or decrease of 0·01 per cent. will show itself in the long run, while each 0·03 per cent. will write its history so that he who runs may read.

"The effect of sulphur upon the cold properties of steel has not been accurately determined, but it is quite certain that it is unimportant. In common practice the content varies from 0·02 to 0·10 per cent., and within these limits it seems to have no appreciable influence upon the elastic ratio, the elongation, or the reduction of area. It is more difficult to say that it does not alter the tensile strength, for a change of 1000 lbs. per square inch can be caused by so many things that it is a bold venture to ascribe it to one variable.

"In rivets, eyebars, and fire-box steel, the presence of sulphur is objectionable, for it will tend to create a coarse crystallization when the metal is heated to a high temperature, and reduce the strength and toughness of the steel.

"In other forms of structural material the effect of this element is probably of little importance."

Another authority[1] states, "the real danger of using a high sulphur steel for structural purposes, even when it has not in any way to be worked hot, lies in the fact that, during rolling, numerous cracks are likely to develop, which close up and are quite imperceptible in the finished material. Nevertheless, these remain as flaws, and may form starting-points for rupture when the material is subjected to any sudden stress. . . . Probably material of this description is one of the most dangerous that can be employed by the engineer, the more so that the tensile strength and elasticity, as evidenced by elongation and reduction of area, will give no indications in the majority of cases that the material is in any way untrustworthy."

"Starting with fairly good materials, with careful treatment manufacturers should have no difficulty in producing regularly a steel with about 0·06 per cent., and certainly 0·08 per cent. is the very maximum that should be allowed in any steel, either for rails or structural purposes."

Influence of Copper.—"Steel may contain up to 1 per cent. of copper without being seriously affected, but if at the same time the sulphur is high, say 0·08 to 0·10 per cent., the cumulative effect is too great for molecular cohesion at high temperatures, and it cracks in rolling. This tearing occurs almost entirely in the first passes of the ingot, so that it is of little importance to the engineer, who is concerned only with perfect finished material. In the purest of soft steels, containing not more than 0·04 per cent. of either phosphorus or sulphur, the influence of even 0·10 per cent. of copper may be detected in the less ready welding of seams during the process of rolling; but ordinarily, when the sulphur is below 0·05 per cent., the copper injures the rolling quality very little, even if present in the proportion of 0·75 per cent. In all cases the cold properties seem to be entirely unaffected.

"These conclusions are not founded on any limited series of tests or special alloys; they are the fruit of years of experience in the making of millions of tons of cupriferous steels, and it is quite certain that any baneful influence of this constant companion would have been felt in the many investigations which have been made into the mechanical equation of structural metal."

Influence of Aluminium.—Experiments by Hadfield quoted by

[1] F. W. Harbord, "The Metallurgy of Steel," 1904.

Campbell show that "after making allowances for the variations in other elements, it will be found that the aluminium has little effect upon the tensile strength, while it does not materially injure the ductility until a content of 2 per cent. is reached."

Experiments by the latter author, however, appear to lead to the following conclusions:—

"(1) The addition of one-half of 1 per cent. of aluminium increases the tensile strength between 3000 and 8000 lbs. per square inch, exalts the elastic limit to about the same proportion, and injures very materially the elongation and contraction of area. The effect both upon strength and ductility is more marked in the case of low than in high steels.

"(2) The addition of another half of 1 per cent. does not have much effect upon the ultimate strength or the elastic limit, but it still further decreases the ductility of the metal."

Influence of Arsenic.—"The effect of arsenic upon steel was quite fully investigated several years ago by Harbord and Tucker. The conclusions given by them may be summarized as follows:—

"Arsenic, in percentages not exceeding 0·17, does not appear to affect the bending properties at ordinary temperatures, but above this percentage cold shortness begins to appear and rapidly increases.

"In amounts not exceeding 0·66 per cent., the tensile strength is raised very considerably. It lowers the elastic limit, and decreases the elongation and reduction of area in a marked degree. It makes the steel harden much more in quenching, and injures its welding power even when only 0·093 per cent. is present.

"These results have been corroborated by J. E. Stead, who found that between 0·10 and 0·15 per cent. of arsenic in structural steel has no material effect upon the mechanical properties; the tenacity is but slightly increased, the elongation and reduction of area apparently unaffected. With 0·20 per cent. of arsenic the difference is noticeable, while with larger amounts the effect is decisive. When 1 per cent. is present, the tenacity is increased, and the elongation and reduction of area both reduced. This increase in strength and diminution in toughness continue as the content of arsenic is raised to 4 per cent., when the elongation and reduction in area become *nil.*"

Influence of Manganese.[1]—"In considering the influence of this

[1] F. W. Harbord, "The Metallurgy of Steel," 1904.

metal on steel, it must be remembered that, unlike most of the other constituents, it is not an impurity originally present which the metallurgical treatment has failed to remove, but is, at all events in the case of all steel used for structural purposes, an essential constituent especially added to deoxidise the decarbonized metal so as to prevent its being red short. The effect which manganese has upon the tenacity and ductility varies very considerably with the percentage of carbon in the steel, its influence being much more marked in the case of high than of low carbon steels. In the author's opinion, for mild steel and rail steel, the less manganese a steel contains above that required to insure solid ingots and freedom from red shortness the better, and with reasonable care taken during the manufacture, there should not be the slightest difficulty in obtaining these results with 0·4 to 0·5 per cent. of manganese in the finished product, at all events for mild steel made in the Siemens furnace. . . . In the case of mild steel required for boiler plates and for bridges or other structural work, an increase of manganese has a very distinct hardening effect, and above 0·6 per cent. begins to be dangerous, and should not be allowed. The tendency amongst steel makers to bring up the tensile strength to the specification by increasing the manganese, instead of the carbon, is greatly to be deprecated, and notwithstanding the reported excellent records of mild steel plates with 1 per cent. of manganese, and steel rails containing more than this amount, engineers will be well advised to decline to accept such material."

It will be evident from a consideration of the foregoing remarks that the relationship between the chemical constitution of mild steel and its ultimate resistance to tension must be of a complex character, and that the attempt to establish a satisfactory formula which shall equate the chemical and physical qualities of any given specimen of the material is surrounded with some difficulties. Several authors have proposed formulæ to this end, but it will suffice here to quote some of the conclusions arrived at by Mr. Campbell as the result of elaborate investigations based on a large number of experiments. For the details and methods employed, the reader is referred to the works of that author.[1]

These conclusions are as follows, converting pounds into tons per square inch :—

[1] "The Manufacture and Properties of Structural Steel;" also the paper read before the Iron and Steel Institute at New York, October, 1904.

The strength of pure iron,[1] as far as it can be determined from the strength of steel, is about 17·76 to 18·75 tons per square inch.

An increase of ·01 per cent. of carbon (determined by combustion) raises the tensile strength of acid steel about 0·44 tons per square inch, and of basic steel about 0·34 tons.

The influence of manganese upon the tensile strength of acid steel is a variable quantity, depending not only upon its own percentage, but upon that of the carbon with which it is associated, and is indicated in the table which follows, for steels of from 0·10 to 0·40 per cent. of carbon.

TABLE No. 2.

ACID STEEL.

Percentage of Carbon.	Increase in tensile strength in tons per square inch corresponding to the percentages of manganese and carbon.										
	Percentage of Manganese.	0·42	0·44	0·46	0·48	0·50	0·52	0·54	0·56	0·58	0·60
0·10		0·07	0·14	0·21	0·28	0·36	0·43	0·50	0·57	0·64	0·71
0·15		0·11	0·22	0·32	0·43	0 53	0·64	0·75	0·86	0·96	1·07
0·20		0·14	0·28	0·42	0·56	0·72	0·86	1·00	1·14	1·28	1·43
0·25		0·18	0·36	0·54	0·72	0·90	1·07	1·25	1·42	1·60	1·78
0·30		0·21	0·42	0·63	0·84	1·08	1·29	1·50	1·71	1 92	2·14
0·35		0·25	0·50	0·75	1·00	1·25	1·50	1·75	2·00	2·25	2·50
0·40		0·28	0·56	0·84	1·14	1·42	1·71	2·00	2·28	2·56	2·85

Thus for a steel of 0·35 per cent. carbon and 0·52 manganese, the increase would be 1·50 tons per square inch.

An increase of 0·01 per cent. of phosphorus raises the tensile strength of acid and basic steel about 0·44 tons per square inch.

The following formulæ give the ultimate strength of acid and basic open-hearth steel in terms of their principal chemical constituents, where $C = 100 \times$ per centage of carbon, $P = 100 \times$ per centage of phosphorus, Mn = manganese, x Mn = a coefficient for manganese in acid steel, of which the values are given in Table No. 2, y Mn = a coefficient manganese in basic steel, of which the values are given in Table No. 3, and R = a variable based on heat treatment.

[1] The term "pure iron" is arbitrary, and intended to express simply the datum plane from which to start in order to find the strength of steel by a simple formula. "Absolutely pure iron never has been, and in all probability never will be, made."

Formula for Acid Open-hearth Steel.
(Carbon estimated by combustion.)

$17{\cdot}85 + 0{\cdot}44\,C + 0{\cdot}44\,P + x\,Mn + R =$ ultimate tensile strength in tons per square inch.

Formula for Acid Open-hearth Steel.
(Carbon estimated by colour.)

$17{\cdot}76 + 0{\cdot}508\,C + 0{\cdot}44\,P + x\,Mn + R =$ ultimate tensile strength in tons per square inch.

Formula for Basic Open-hearth Steel.
(Carbon estimated by combustion.)

$18{\cdot}52 + 0{\cdot}34\,C + 0{\cdot}44\,P + y\,Mn + R =$ ultimate tensile strength in tons per square inch.

Formula for Basic Open-hearth Steel.
(Carbon estimated by colour.)

$18{\cdot}75 + 0{\cdot}366\,C + 0{\cdot}44\,P + y\,Mn + R =$ ultimate tensile strength in tons per square inch.

In the above formula, R, the variable for heat treatment, is zero, in angles and plates about $\frac{3}{8}$ inch to $\frac{1}{2}$ inch thick finished at a fairly high temperature.

The influence of manganese upon the tensile strength of basic steel is given in the following table :—

TABLE No. 3.

BASIC STEEL.

Percentage of carbon.	Increase in tensile strength in tons per square inch corresponding to the percentages of manganese and carbon.						
	Percentage of manganese.	0·35	0·40	0·45	0·50	0·55	0·60
0·05		0·24	0·49	0·73	0·98	1·22	1·47
0·10		0·29	0·58	0·86	1·16	1·44	1·74
0·15		0·33	0·66	1·00	1·33	1·66	2·00
0·20		0·37	0·75	1·13	1·51	1·89	2·27
0·25		0·42	0·84	1·26	1·69	2·11	2·54
0·30		0·46	0·93	1·39	1·87	2·33	2·81
0·35		0·51	1·02	1·53	2·05	2·56	3·08
0·40		0·56	1·12	1·67	2·23	2·79	3·35

As an example of the application of the above formulæ, let us assume a specimen of open-hearth acid steel of which the chemical analysis gives a percentage of carbon (estimated by combustion) of 0·166, phosphorus 0·053, manganese 0·58; then by the formula we have—

$$\left.\begin{array}{l}\text{Ultimate tensile}\\ \text{strength in tons}\\ \text{per square inch}\end{array}\right\} = 17{\cdot}85 + (0{\cdot}44 \times 100 \times 0{\cdot}166) + (0{\cdot}44 \times 100 \times 0{\cdot}053) + 1{\cdot}00 \text{ (see Table No. 2)} = 28{\cdot}5 \text{ tons.}$$

In steels containing less than 0·25 per cent. of carbon, the effect of small proportions of silicon upon the ultimate strength is inappreciable.

Sulphur in ordinary proportions exerts no appreciable influence upon the tensile strength.

It will be observed, from a comparison of the above formulæ, that phosphorus causes an addition to the tensile strength for each 0·01 per cent. equal to that caused by carbon for each 0·01 per cent., and this consideration gives force to Mr. Campbell's remark that "it is well not to assume the truth of all tradition, but if there is one fact which seems demonstrated, it is that phosphorus will hide its true character in the testing machine, but will certainly make itself known at some future time."

The great bulk of the material known as mild steel is, in Europe and America, produced by the following processes, viz.:—

The Acid Bessemer process;
The Basic Bessemer process;
The Acid Open-hearth process;
The Basic Open-hearth process.

The process by which high carbon steels are produced, known as the "Crucible," need not here be further alluded to, as the quality of steel produced by this method is not that used in those forms of construction with which this work principally deals, being, in fact, chiefly used in the manufacture of machine tools and implements, cutting instruments of keen temper and fine edge, and for other similar purposes.

With regard to the above-mentioned processes, it will be observed that they consist of two principal divisions, viz. the Bessemer and open-hearth (otherwise the Siemens or Siemens-Martin process), each division being further subdivided into the processes known as acid and basic.

The authority previously quoted[1] has defined each of these methods of manufacture in general terms, as follows:—

"The acid Bessemer process consists in blowing air into liquid pig-iron for the purpose of burning most of the silicon, manganese, and carbon of the metal, the operation being conducted in an acid-lined vessel, and in such a manner that the product is entirely fluid."

"The basic Bessemer process consists in blowing air into liquid pig-iron for the purpose of burning most of the silicon, manganese, carbon, phosphorus, and sulphur of the metal, the operation being conducted in a basic-lined vessel, and in such a manner that the product is entirely fluid."

"The open-hearth process consists in melting pig-iron, mixed with more or less wrought-iron, steel, or similar iron products, by exposure to the direct action of the flame in a regenerative gas furnace and converting the resultant bath into steel, the operation being so conducted that the final product is entirely fluid."

We have seen that the open-hearth process may be either acid or basic. Of the latter the same author says—

"The basic (open-hearth) process, as herein discussed, consists in melting a charge of pig-iron, or a mixture of pig-iron and low carbon metal upon a hearth of dolomite, lime, magnetite, or other basic or passive material, and converting it into steel in the presence of a stable basic slag by the action of the flame, with or without the use of ore, and by the addition of the proper recarbonizers, the operation being so conducted that the product is cast in a fluid state."

Amplifying the above general description, the following essential points of difference may be noted. In the Bessemer process the high temperature required for combustion and to effect the necessary chemical changes is maintained by blowing air through the molten pig-iron.

In the open hearth no such blowing through process takes place, the bath of steel being exposed to the direct influence of the flame and intense temperature produced by the use of the Siemens regenerative furnace, which forms an essential feature in this method of manufacture.

When we next consider the essential differences which underlie the use of the terms "acid" and "basic," we find, however, points

[1] H. H. Campbell, "The Manufacture and Properties of Structural Steel."

of detail which are of importance as regards the quality of the resultant material.

The influences, mainly hostile, exerted by the elements of phosphorus and sulphur, but more especially the former, upon the physical qualities of the finished product have already been enlarged upon in the foregoing remarks.

The extent, therefore, to which the elimination of these hostile influences can be carried by the various processes of steel manufacture, having regard to the original quality of the ore used, must consequently claim our attention, if we are to make any selection as to the method by which the finished product desired is to be manufactured.

It is beyond the scope of these notes to enter fully into the complete history of the changes which take place in the contents of the acid-lined Bessemer converter from the commencement to the end of the "blow." Suffice it to say that while the original carbon content has been burnt out until practically none is left, the ultimate desired percentage of carbon being obtained by recarbonization by means of the addition of spiegel or ferro manganese, the element of phosphorus remains at nearly the same percentage as that in the original stock of molten pig-iron or scrap at the commencement of the blow.

For a given percentage of phosphorus in the finished product, it follows therefore that the original stock must contain no more phosphorus than that allowed at the finish.

This implies the use of practically non-phosphoric ores for the acid process.

The acid-lined open hearth in this respect stands on the same footing as the acid-lined converter, and the original stock must be of known composition so far as sulphur and phosphorus are concerned, for there is no appreciable elimination of these elements, and the finished product will show a percentage equal to the average of the material charged.

In the basic Bessemer process the distinctive feature of the basic vessel is a lining which resists the action of basic slags. This is usually made of dolomite, or limestone containing a small proportion of magnesia.

During the earlier stages of the process of combustion the chemical reactions in the metal of the basic converter are practically identical with the reactions in the acid vessel up to the point when the combustion of the carbon has been carried to

its limit. From this point onwards to the end of the blow the comparison with the acid process ceases, and the distinctive feature of the basic system, viz. the combustion of the phosphorus and sulphur, begins. The initial content of phosphorus can be burnt out and reduced to a desirable limit.

This dephosphorization is in a similar manner the characteristic feature of the basic open hearth as compared with the acid open hearth.

In both these basic processes it is, then, possible to use an initial stock of pig or scrap having a higher percentage of the undesirable elements than is possible in the acid processes; or, in other words, a less pure ore can be utilized.

An important distinction between the converter and open-hearth system lies in the fact that whereas the initial charge of pig-iron or scrap can be converted into steel by the former process in from fifteen to twenty minutes, the same transformation by the latter process occupies some nine to twelve hours. In the opinion of many authorities, this difference of time exercises an important influence on the quality of the resulting material by reason of the fact that greater opportunities are afforded in the longer process of testing the quality at frequent stages of the process.[1]

The important question, by which of the processes can the best and most reliable mild steel be produced for structural purposes, is one which would probably be answered by British, American, or German steel makers from points of view not wholly unconnected with the great commercial interests involved in the supply and use, in their respective countries, of phosphoric or non-phosphoric ores.

It may, however, be generally admitted that for uniformity of quality, and general excellence of material for all purposes where great reliability is essential, the product of the open hearth, either acid or basic, stands pre-eminent.

In support of this view a series of tests is appended, representative of present-day open-hearth practice in this country, and similar tests might be multiplied almost indefinitely.

The tests cover, it will be seen, a large range of sections of structural material, such as are commonly employed in every-day use, and they have been exhibited at some length in order that

[1] Various modifications of the open-hearth process (involving the consideration of various points of steel works practice into which it is not necessary here to enter) are known as the Bertrand-Thiel process, the Talbot process, the Twynam process, and the Monell process.

the student may be assured of the practical application of the series to the work he may have under consideration.

The tests are the samples of a large quantity of mild steel employed in ordinary structural work as represented and described in Chapters III. to VI., and are representatives of the material from which the majority of the girder-work, columns, roofing, etc., represented by the illustrations in this volume, have been manufactured.[1]

The material was supplied under ordinary commercial conditions by some seven or eight firms in both England, Wales, and Scotland, and are therefore fairly indicative of present practice in open-hearth work in Great Britain. The material was supplied under the following specification—

All mild steel required for structural purposes is to be of British manufacture, made by the open-hearth process, either acid or basic.

To be cleanly rolled and true to the thicknesses and sections specified, free from scale, laminations, cracked edges, and every other defect.

The edges of all plates to be cleanly sheared, except where otherwise specified, and truly square. The surfaces of finished plates to be quite fair and flat, except where otherwise directed.

All steel to be of such strength and quality that it shall not fracture under tensile stresses or with elongations less than those shown in the following table:—

Description of material.	Tensile strength in tons per square inch.		Elongation in 8-inch length.
	Not less than	Not more than	
			Per cent.
Rivet and bolt steel	26	30	25
Strips cut lengthwise from beams, angles, tees, channels, and bars, both square and round	26	30	20
Strips cut lengthwise or crosswise from plates	26	30	20

[1] These tests, together with the chemical analyses, were carried out by Mr. R. H. Harry Stanger, Assoc. M. Inst. C. E., A. M. I. Mech. E., Broadway Testing Works, Westminster.

Samples selected for testing as specified above are to be planed parallel for a length of 8 inches. The sectional area to be fractured is, whenever possible, to be not less than ½ square inch. The steel must also be capable of bearing the following tests :—

Rivets.—Pieces of rivet steel, heated uniformly to a low cherry red, and cooled in water of 82° Fahrenheit, must stand bending double in a press to a curve of which the inner diameter is equal to the diameter of the bar tested.

Bending cold without fracture in the manner shown in Fig. 1, where the line AB equals one diameter of the rivet.

Bending double when hot, and hammered till the two parts of the shank touch in the manner shown in Fig. 2 without fracture.

Flattening of the rivet head while hot in the manner shown in Fig. 3 without cracking at the edges. The head to be flattened

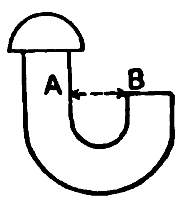


Fig. 1.

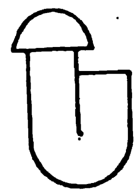


Fig. 2.

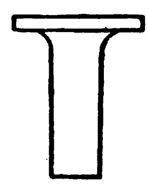


Fig. 3.

until its diameter is two and a half times the diameter of the shank.

The shank of the rivet to be nicked on one side, and bent over to show the quality of the material.

Bolts and Nuts.—Pieces cut from a bar, heated uniformly to a low cherry red and cooled in water at 80° Fahrenheit, must stand bending in a press to a curve of which the inner radius is equal to the radius of the bar tested.

A sample bolt is to be slightly notched and bent over to show the quality of the material.

When the bolts are of sufficient length in the plain part to admit of being bent cold, they must stand bending in a press to a curve of which the inner radius is equal to the radius of the bolt tested without fracture.

When the bolts are not of sufficient length in the plain part to admit of being bent cold, the screwed part should stand bending cold without fracture, as follows :—

	½ inch diameter, and under,	through an angle of			35°
above ½ inch	„	and under 1 inch	„	„	30°
1 inch	„	and above	„	„	25°

Beams, angles, channels, tees, etc.—Strips cut lengthwise, 1½ in. wide, heated uniformly to a low cherry red and cooled in water of about 80° Fahrenheit, must stand bending double in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested.

This steel is also to stand such forge tests, both hot and cold, as may be sufficient in the opinion of the inspector to prove soundness of material and fitness for the work.

Plates.—Strips cut lengthwise or crosswise, 1½ inch wide, heated uniformly to a low cherry red, and cooled in water of about 80° Fahrenheit, must stand bending double in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested. Such other tests as may be considered necessary by the inspector to determine the quality of the steel plates are also to be carried out. Samples will be taken as often and in such a manner as the inspector may consider necessary, and in the event of a sample proving unsatisfactory, it will be in the power of the inspector to reject the whole of the steel represented by such sample.

Steel Castings.—Steel castings to be sound, true, and clean, and free from honeycomb.

Pieces of 1 inch square, taken from each cast or blow of steel, to have a breaking strain of 26 tons per square inch, with an elongation of not less than 10 per cent. in a length of 8 inches. A test piece, 1 inch square, shall be capable of bending cold in a press or over a slab or block with a fair surface, with the edge with a rounding of 1½ inch radius, through an angle of 45°.

The castings to be thoroughly annealed by being put in a special furnace and carefully heated up to a bright cherry-red and then allowed to cool gradually. The castings are not to be taken out of the furnace until sufficiently cool to admit of them being easily handled without covering. The duration of time from heating to cooling to be not less than seven days.

The castings to be afterwards slung and tested by hammering to ensure soundness.

All castings to be chipped and dressed to remove roughness or inequalities.

All test pieces required are to be properly shaped and prepared for testing at the contractors' cost.

TABLES OF THE RESULTS OF PHYSICAL TESTS ON THE ULTIMATE TENSILE STRENGTH AND ELONGATION OF OPEN-HEARTH MILD STEEL FOR ORDINARY STRUCTURAL WORK.

TABLE No. 4.

Tests on Mild Steel Angles.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel angles.			
1	9″ × 3½″ × ½″	28·0	26·0	Bending tests satisfactory
2	8″ × 3½″ × ⅝″	30·2	22·0	” ”
3	” ”	29·1	26·0	” ”
4	” ”	30·0	29·0	” ”
5	7″ × 4″ × ⅝″	29·9	27·0	” ”
6	” ”	28·7	28·0	” ”
7	7″ × 3½″ × ⅝″	28·7	26·0	” ”
8	7″ × 3½″ × ½″	28·5	25·0	” ”
9	7″ × 3″ × ⅜″	30·8	25·0	Excess strength slight, elongation good, so allowed
10	” ”	30·7	26·0	
11	6½″ × 4½″ × ⅝″	30·1	25·0	Bending tests satisfactory
12	6″ × 6″ × 11/16″	27·2	23·0	” ”
13	6″ × 6″ × ⅝″	26·6	23·0	” ”
14	6″ × 4″ × ½″	28·1	26·0	” ”
15	” ”	28·6	24·0	” ”
16	6″ × 3½″ × ½″	27·32	29·5	” ”
17	6″ × 3″ × ½″	26·3	23·0	” ”
18	” ”	28·1	27·5	” ”
19	” ”	31·3	23·0	Somewhat above the specified maximum, but the elongation being good, the test was allowed
20	6″ × 3″ × ⅜″	28·7	27·0	Bending tests satisfactory
21	” ”	28·7	27·0	” ”
22	” ”	28·6	24·0	” ”
23	” ”	28·4	22·0	” ”
24	5″ × 5″ × ⅝″	27·9	27·0	” ”
25	5″ × 5″ × ½″	29·7	25·0	” ”
26	5″ × 3½″ × ½″	29·9	25·0	” ”
27	5″ × 3½″ × ⅜″	29·8	25·0	” ”
28	” ”	29·6	23·0	” ”

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel angles.			
29	$5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	29·8	28·0	Bending tests satisfactory
30	,, ,,	28·6	27·0	,, ,,
31	,, ,,	28·1	24·0	,, ,,
32	,, ,,	29·2	25·0	,, ,,
33	,, ,,	29·5	27·0	,, ,,
34	,, ,,	29·4	24·0	,, ,,
35	$5'' \times 3'' \times \frac{3}{8}''$	29·4	26·0	,, ,,
36	,, ,,	29·6	26·0	,, ,,
37	$4\frac{1}{2}'' \times 4'' \times \frac{5}{8}''$	28·6	25·0	,, ,,
38	,, ,,	28·7	24·0	,, ,,
39	$4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	26·8	28·0	,, ,,
40	$4\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	29·1	29·0	,, ,,
41	,, ,,	29·3	26·0	,, ,,
42	$4'' \times 4'' \times \frac{5}{8}''$	27·7	29·0	,, ,,
43	,, ,,	30·3	24·0	,, ,,
44	,, ,,	27·4	32·0	,, ,,
45	$4'' \times 4'' \times \frac{1}{2}''$	28·2	30·0	,, ,,
46	,, ,,	29·7	28·0	,, ,,
47	$4'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	28·9	26·0	,, ,,
48	$4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	29·9	27·0	,, ,,
49	,, ,,	28·2	26·0	,, ,,
50	$4'' \times 3'' \times \frac{5}{8}''$	29·2	23·0	,, ,,
51	$4'' \times 3'' \times \frac{1}{2}''$	27·5	25·5	,, ,,
52	,, ,,	27·7	26·0	,, ,,
53	,, ,,	27·9	21·0	,, ,,
54	,, ,,	27·4	34·0	,, ,,
55	,, ,,	27·7	26·0	,, ,,
56	,, ,,	28·0	23·0	,, ,,
57	,, ,,	28·2	27·0	,, ,,
58	,, ,,	27·8	24·0	,, ,,
59	,, ,,	28·8	27·0	,, ,,
60	,, ,,	30·0	25·0	,, ,,
61	$4'' \times 3'' \times \frac{3}{8}''$	28·3	25·0	,, ,,
62	,, ,,	28·2	29·0	,, ,,
63	,, ,,	28·9	25·0	,, ,,
64	,, ,,	29·7	23·5	,, ,,
65	,, ,,	27·7	29·0	,, ,,
66	,, ,,	28·2	27·0	,, ,,
67	,, ,,	27·8	29·0	,, ,,
68	$4'' \times 3'' \times \frac{5}{16}''$	29·0	21·5	,, ,,
69	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	29·8	27·0	,, ,,
70	,, ,,	29·1	25·0	,, ,,
71	,, ,,	28·7	29·0	,, ,,
72	,, ,,	27·7	29·0	,, ,,

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel angles.			
73	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	27·8	26·0	Bending tests satisfactory
74	„ „	28·7	29·0	„ „
75	„ „	27·7	29·0	„ „
76	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	29·0	26·5	„ „
77	„ „	27·8	30·0	„ „
78	„ „	28·2	32·0	„ „
79	„ „	28·0	25·0	„ „
80	„ „	26·1	30·0	„ „
81	„ „	26·5	30·0	„ „
82	„ „	27·4	28·0	„ „
83	„ „	26·3	28·0	„ „
84	„ „	26·7	31·0	„ „
85	„ „	26·6	28·5	„ „
86	„ „	26·7	28·5	„ „
87	„ „	27·6	32·0	„ „
88	„ „	28·6	25·0	„ „
89	„ „	28·9	27·5	„ „
90	„ „	30·0	25·0	„ „
91	„ „	29·9	28·0	„ „
92	„ „	28·7	27·0	„ „
93	„ „	29·3	26·0	„ „
94	„ „	29·1	25·0	„ „
95	„ „	30·0	24·0	„ „
96	„ „	29·1	27·0	„ „
97	„ „	30·1	25·0	Excess strength slight, elongation good, so allowed
98	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	30·3	25·0	Round backed
99	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	27·6	27·0	Bending tests satisfactory
100	„ „	28·5	26·0	„ „
101	„ „	28·2	27·0	„ „
102	„ „	28·4	28·0	„ „
103	$3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	28·5	24·0	„ „
104	„ „	30·2	24·0	„ „
105	„ „	28·1	28·0	„ „
106	$3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	29·1	26·0	„ „
107	„ „	29·1	26·0	„ „
108	$3'' \times 3'' \times \frac{1}{2}''$	27·0	23·0	„ „
109	„ „	26·5	29·0	„ „
110	„ „	29·2	27·0	„ „
111	„ „	30·7	27·0	Excess strength slight, elongation good, so allowed
112	„ „	30·9	25·0	
113	„ „	27·9	25·5	Bending tests satisfactory

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel angles.			
114	$3'' \times 3'' \times \frac{1}{2}''$	28·7	27·5	Bending tests satisfactory
115	" "	27·2	27·0	" "
116	" "	27·7	26·0	" "
117	" "	29·4	27·0	" "
118	$3'' \times 3'' \times \frac{3}{8}''$	27·2	26·0	" "
119	" "	28·2	28·0	" "
120	" "	28·9	29·0	" "
121	" "	29·8	29·0	" "
122	" "	27·5	28·0	" "
123	" "	28·5	25·0	" "
124	" "	29·1	27·0	" "
125	$3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	29·8	25·0	" "
126	" "	26·8	25·0	" "
127	" "	26·4	22·5	" "
128	$3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$	26·1	33·0	" "
129	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	28·4	26·0	" "
130	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	27·0	29·0	" "
131	" "	27·3	30·0	" "
132	" "	29·7	27·2	" "
133	" "	27·0	25·0	" "
134	" "	28·9	26·0	" "
135	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$	29·4	21·0	" "
136	" "	28·9	22·0	" "
137	" "	28·7	23·0	" "
138	" "	29·6	20·0	" "
139	" "	30·0	28·0	" "
140	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	28·9	22·0	" "
141	$2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{3}{8}''$	29·0	20·0	" "
142	" "	29·3	25·0	" "
143	" "	28·3	24·0	" "
144	" "	28·1	24·0	" "
145	$2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{5}{16}''$	28·6	21·0	" "
146	" "	27·5	22·0	" "
147	" "	28·7	21·0	" "
148	" "	27·3	25·0	" "
149	" "	28·8	22·0	" "
150	$2'' \times 2'' \times \frac{5}{16}''$	31·1	22·0	Excess strength, but elongation good, so allowed
151	" "	26·0	30·0	Bending tests satisfactory
152	$2'' \times 1\frac{1}{2}'' \times \frac{3}{8}''$	30·7	20·0	Excess strength slight, elongation good, so allowed

TABLE No. 5.

Tests on Mild Steel Tees.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel tees.			
1	6″ × 4″ × $\frac{1}{2}$″	27·6	26·0	Bending tests satisfactory
2	6″ × 3″ × $\frac{1}{2}$″	28·6	27·0	,, ,,
3	,, ,,	27·4	29·0	,, ,,
4	,, ,,	28·1	27·5	,, ,,
5	,, ,,	31·3	23·0	Somewhat above the specified maximum, but the elongation being good the test was allowed.
6	,, ,,	30·5	28·0	
7	,, ,,	26·85	31·5	Bending tests satisfactory
8	,, ,,	29·7	27·0	,, ,,
9	,, ,,	29·3	25·0	,, ,,
10	,, ,,	29·4	28·0	,, ,,
11	,, ,,	31·4	22·0	Somewhat above the specified maximum, but the elongation being good the test was allowed.
12	,, ,,	29·5	31·0	Bending tests satisfactory
13	,, ,,	28·6	31·0	,, ,,
14	,, ,,	27·6	32·0	,, ,,
15	,, ,,	27·6	25·0	,, ,,
16	6″ × 3″ × $\frac{3}{8}$″	30·7	25·0	Somewhat above the specified maximum, but the elongation being good the test was allowed.
17	,, ,,	27·4	27·5	Bending tests satisfactory
18	,, ,,	29·5	26·0	,, ,,
19	,, ,,	29·6	27·0	,, ,,
20	,, ,,	27·1	29·0	,, ,,
21	5″ × 3″ × $\frac{3}{8}$″	29·3	29·0	,, ,,
22	,, ,,	29·3	26·0	,, ,,
23	,, ,,	29·3	31·0	,, ,,
24	,, ,,	29·5	28·5	,, ,,
25	5″ × $2\frac{1}{2}$″ × $\frac{3}{8}$″	28·7	27·0	,, ,,
26	,, ,,	29·7	25·0	,, ,,
27	,, ,,	27·3	31·0	,, ,,
28	,, ,,	28·5	28·0	,, ,,
29	,, ,,	29·4	29·0	,, ,,

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel tees.			
30	5″ × 2½″ × ⅜″	29·2	21·5	Bending tests satisfactory
31	4″ × 4″ × ½″	29·7	28·5	,, ,,
32	,, ,,	28·8	29·5	,, ,,
33	,, ,,	28·9	28·0	,, ,,
34	,, ,,	29·3	30·0	,, ,,
35	,, ,,	28·2	27·0	,, ,,
36	,, ,,	29·5	35·0	,, ,,
37	,, ,,	27·8	25·0	,, ,,
38	,, ,,	27·3	32·0	,, ,,
39	,, ,,	27·5	33·0	,, ,,
40	4″ × 4″ × ⅜″	29·7	26·0	,, ,,
41	,, ,,	29·3	27·5	,, ,,
42	,, ,,	28·9	28·0	,, ,,
43	,, ,,	27·7	28·0	,, ,,
44	,, ,,	29·1	27·5	,, ,,
45	,, ,,	27·3	30·0	,, ,,
46	,, ,,	27·5	32·0	,, ,,
47	,, ,,	27·1	27·0	,, ,,
48	,, ,,	27·6	28·0	,, ,,
49	,, ,,	27·7	28 0	,, ,,
50	,, ,,	27·6	27·0	,, ,,
51	,, ,,	27·6	27·0	,, ,,
52	,, ,,	28·6	24·0	,, ,,
53	,, ,,	29·2	27·0	,, ,,
54	,, ,,	30·3	22·0	,, ,,
55	,, ,,	29·4	25·0	,, ,,
56	4″ × 3½″ × ½″	28·3	26·0	,, ,,
57	4″ × 3″ × ⅜″	29·3	27·5	,, ,,
58	2½″ × 2½″ × ⅜″	27·2	27·5	,, ,,
59	2″ × 1½″ × $\frac{5}{16}$″	28·7	15·0	Broke on the datum point
60	,, ,,	30·0	21·5	The following test, No. 60, quite satisfactory, bending tests satisfactory

TABLE No. 6.

Tests on Mild Steel Flats.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per square inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel flats.			
1	$18\frac{1}{4}'' \times \frac{1}{2}''$	29·4	29·0	Bending tests satisfactory
2	„	29·2	29·5	„ „
3	$16'' \times \frac{1}{2}''$	28·9	29·0	„ „
4	„	28·8	28·0	„ „
5	„	27·7	28·0	„ „
6	„	28·0	30·0	„ „
7	$16'' \times \frac{3}{8}''$	29·0	27·0	„ „
8	„	29·7	27·0	„ „
9	„	28·6	25·0	„ „
10	„	28·0	29·0	„ „
11	$14'' \times \frac{5}{8}''$	30·5	26·0	„ „
12	„	27·8	28·0	„ „
13	$14'' \times \frac{1}{2}''$	27·9	29·0	„ „
14	„	28·7	26·0	„ „
15	„	29·7	26·0	„ „
16	„	28·9	29·0	„ „
17	„	28·9	28·0	„ „
18	„	29·7	27·0	„ „
19	$14'' \times \frac{3}{8}''$	28·8	28·0	„ „
20	$12'' \times \frac{5}{8}''$	28·5	28·0	„ „
21	„	29·8	25·0	„ „
22	$12'' \times \frac{9}{16}''$	27·4	28·0	„ „
23	$12'' \times \frac{1}{2}''$	31·2	23·0	Although the tensile strength is above that specified, the elongation and bending tests are satisfactory. The material was accepted
24	$12'' \times \frac{1}{2}''$	25·2	28·0	Tensile strength below that specified, but further tests were satisfactory, so allowed
25	$12'' \times \frac{1}{2}''$	28·1	28·5	Bending tests satisfactory
26	„	26·0	30·0	„ „
27	„	28·7	29·0	„ „
28	„	27·9	25·5	„ „
29	„	25·8	29·0	„ „

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per square inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel flats.			
30	12″ × $\frac{3}{8}$″	31·3	21·0	Although the tensile strength is above that specified, the elongation and bending tests are satisfactory. Material was accepted
31	11″ × $\frac{5}{8}$″	27·2	28·5	Bending tests satisfactory
32	11″ × $\frac{1}{2}$″	27·0	32·0	,, ,,
33	,,	28·1	31·0	,, ,,
34	,,	29·5	30·0	,, ,,
35	,,	29·3	27·0	,, ,,
36	10″ × $\frac{5}{8}$″	30·0	29·0	,, ,,
37	10″ × $\frac{3}{8}$″	29·5	28·0	,, ,,
38	,,	29·9	29·0	,, ,,
39	,,	28·7	26·0	,, ,,
40	,,	28·5	28·0	,, ,,
41	,,	28·4	27·0	,, ,,
42	9″ × 1″	29·5	21·0	,, ,,
43	9″ × $\frac{5}{8}$″	30·6	29·0	,, ,,
44	9″ × $\frac{1}{2}$″	28·3	27·0	,, ,,
45	9″ × $\frac{3}{8}$″	28·6	25 0	,, ,,
46	8$\frac{1}{2}$ × $\frac{3}{4}$″	27·2	27·0	,, ,,
47	,,	27·4	29·0	,, ,,
48	8$\frac{1}{2}$″ × $\frac{5}{8}$″	29·0	26·5	,, ,,
49	,,	27·4	29·0	,, ,,
50	8$\frac{1}{2}$″ × $\frac{1}{2}$″	29·0	26·0	,, ,,
51	,,	30·0	30·0	,, ,,
52	,,	30·4	26·0	,, ,,
53	8$\frac{1}{2}$″ × $\frac{5}{16}$″	29·6	29·0	,, ,,
54	8$\frac{1}{4}$″ × $\frac{1}{2}$″	28·0	30·0	,, ,,
55	8″ × $\frac{1}{2}$″	27·2	24·0	,, ,,
56	,,	26·6	24·0	,, ,,
57	8″ × $\frac{5}{16}$″	30·5	26·0	,, ,,
58	7$\frac{1}{2}$″ × $\frac{3}{4}$″	27·0	31·0	,, ,,
59	7$\frac{1}{2}$″ × $\frac{1}{2}$″	27·0	31·0	,, ,,
60	7″ × $\frac{7}{8}$″	27·2	26·0	,, ,,
61	7″ × $\frac{1}{2}$″	28·5	29·0	,, ,,
62	7″ × $\frac{3}{8}$″	28·6	33·0	,, ,,
63	,,	28·5	28·0	,, ,,
64	,,	29·7	22·0	,, ,,
65	,,	30·2	27·0	,, ,,
66	6$\frac{1}{2}$″ × $\frac{1}{2}$″	26·7	28·0	,, ,,
67	6″ × $\frac{5}{8}$″	28·8	28·0	,, ,,
68	6″ × $\frac{1}{2}$″	26·7	31·0	,, ,,

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per square inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel flats.			
69	$5\frac{1}{2}'' \times \frac{1}{2}''$	27·2	30·0	Bending tests satisfactory
70	$5\frac{1}{2}'' \times \frac{7}{16}''$	27·7	27·0	" "
71	$5'' \times 1''$	27·6	25·0	" "
72	$5'' \times \frac{1}{2}''$	28·8	26·0	" "
73	$5'' \times \frac{5}{16}''$	31·2	20·0	The forge tests being satisfactory, and, having in view the thinness of the bar, the material was accepted
74	$5'' \times \frac{1}{4}''$	27·1	23·0	Bending tests satisfactory
75	$4'' \times \frac{7}{8}''$	28·5	25·0	" "
76	$4'' \times \frac{1}{2}''$	29·7	24·0	" "
77	$3\frac{1}{2}'' \times 1\frac{1}{4}''$	30·0	21·0	" "
78	$3\frac{1}{2}'' \times 1\frac{1}{8}''$	28·4	27·0	" "
79	$3\frac{1}{2}'' \times \frac{1}{2}''$	28·0	28·0	" "
80	"	28·2	25·0	" "
81	"	28·2	24·0	" "
82	"	30·0	23·0	" "
83	"	28·6	28·0	" "
84	"	28·7	25·0	" "
85	"	27·8	28·0	" "
86	"	27·7	26·0	" "
87	"	28·6	24·0	" "
88	$3\frac{1}{2}'' \times \frac{3}{8}''$	29·7	22·0	" "
89	$3'' \times 1''$	28·3	28·0	" "
90	$3'' \times \frac{7}{8}''$	26·4	25·0	" "
91	$3'' \times \frac{1}{2}''$	29·2	24·0	" "
92	$3'' \times \frac{3}{8}''$	29·5	25·0	" "
93	"	29·4	23·0	" "
94	"	28·6	22·0	" "
95	"	29·1	21·0	" "
96	"	29·9	20·0	" "
97	$3'' \times \frac{1}{4}''$	29·0	24·0	" "
98	"	28·6	28·0	" "
99	$2\frac{1}{2}'' \times \frac{3}{4}''$	27·7	23·0	" "
100	$2\frac{1}{2}'' \times \frac{5}{8}''$	29·3	21·0	" "
101	$2\frac{1}{2}'' \times \frac{1}{2}''$	30·1	20·0	" "
102	$2\frac{1}{2}'' \times \frac{3}{8}''$	26·0	25·0	" "
103	"	31·4	31·0	Somewhat above the specified maximum, but the elongation being good the test was allowed
104	$2\frac{1}{2}'' \times \frac{5}{16}''$	27·9	26·0	Bending tests satisfactory
105	$2\frac{1}{4}'' \times \frac{1}{2}''$	27·8	24·0	" "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per square inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel flats.			
106	2" × ½"	29·0	28·0	Bending tests satisfactory
107	2" × ⅜"	28·1	27·0	,, ,,
108	,,	26·7	27·0	,, ,,
109	1¾" × 7/16"	28·2	27·0	,, ,,

TABLE No. 7.

Tests on Mild Steel Channels.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel channels.			
1	15" × 4" @ 42 lbs.	29·3	25·0	Bending tests satisfactory
2	12" × 3½" @ 31 lbs.	29·3	24·0	,, ,,
3	10" × 4 × ⅝	26·6	24·0	,, ,,
4	,, ,,	27·0	25·0	,, ,,
5	10" × 4" × ½"	30·3	30·0	,, ,,
6	10" × 3" @ 26 lbs.	27·8	28·0	,, ,,
7	,, ,,	28·3	26·0	,, ,,
8	9" × 3½ × ½	29·5	26·0	,, ,,
9	,, ,,	28·2	26·0	,, ,,
10	9" × 3" @ 15·45 lbs.	29·9	25·0	,, ,,
11	,, ,,	30·1	26·0	,, ,,
12	,, ,,	29·8	27·0	,, ,,
13	,, ,,	29·5	27·0	,, ,,
14	,, ,,	29·6	27·0	,, ,,
15	,, ,,	29·5	25·0	,, ,,
16	,, ,,	29·4	25·0	,, ,,
17	8" × 3½ × ½"	30·2	22·0	,, ,,
18	,, ,,	29·5	25·0	,, ,,
19	7⅞" × 2⅞" × 20¼ lbs.	26·3	30·0	,, ,,
20	7⅞" × 2¾"	28·3	21·0	,, ,,
21	,,	28·7	24·0	,, ,,
22	,,	28·1	23·0	,, ,,
23	7⅞" × 2½" × 15¼ lbs.	26·0	28·0	,, ,,
24	,, ,,	29·0	24·0	,, ,,

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel channels.			
25	7″ × 3½″ × ½″	29·1	27·0	Bending tests satisfactory
26	” ”	29·8	22·0	” ”
27	” ”	29·0	26·0	” ”
28	” ”	28·3	24·0	” ”
29	” ”	29·5	24·0	” ”
30	” ”	28·8	28·0	” ”
31	” ”	29·6	21·0	” ”
32	” ”	28·3	25·0	” ”
33	” ”	30·0	23·0	” ”
34	7″ × $2\frac{15}{16}$″ × $\frac{7}{16}$″ × ½″	28·1	23·0	” ”
35	6″ × 3″ × ½″	28·7	29·0	” ”

TABLE No. 8.

Tests on Mild Steel Rolled Joists.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel rolled joists.			
1	16″ × 6″ @ 62 lbs.	28·6	30·0	Bending tests satisfactory
2	14″ × 6″ @ 57 lbs.	30·6	25·0	” ”
3	” ”	28·2	27·0	” ”
4	14″ × 6″ @ 46 lbs.	27·4	21·0	” ”
5	” ”	27·4	30·0	” ”
6	13″ × 5″ @ 40 lbs.	29·7	22·0	” ”
7	” ”	29·4	24·0	” ”
8	” ”	29·6	23·0	” ”
9	” ”	29·6	22·0	” ”
10	12″ × 6″ @ 54 lbs.	29·7	23·0	” ”
11	” ”	28·4	24·0	” ”
12	” ”	28·3	25·0	” ”
13	” ”	28·0	22·0	” ”
14	” ”	28·8	28·0	” ”
15	” ”	28·9	28·0	” ”
16	” ”	28·8	24·0	” ”
17	” ”	28·6	21·0	” ”

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel rolled joists.			
18	12″ × 6″ @ 54 lbs.	29·3	28·0	Bending tests satisfactory
19	,, ,,	28·3	22·0	,, ,,
20	,, ,,	29·4	24·0	,, ,,
21	,, ,,	29·0	24·0	,, ,,
22	,, ,,	28·0	28·0	,, ,,
23	12″ × 6″ @ 44 lbs.	28·2	29·0	,, ,,
24	,, ,,	28·6	29·0	,, ,,
25	12″ × 5″ @ 32 lbs.	28·6	26·0	,, ,,
26	,, ,,	28·2	23·0	,, ,,
27	,, ,,	27·7	25·0	,, ,,
28	,, ,,	28·8	28·0	,, ,,
29	10″ × 6″ @ 45 lbs.	27·7	31·0	,, ,,
30	,, ,,	27·7	29·0	,, ,,
31	,, ,,	28·8	30·0	,, ,,
32	,, ,,	29·1	30·0	,, ,,
33	,, ,,	27·8	29·0	,, ,,
34	,, ,,	28·1	35·0	,, ,,
35	,, ,,	28·2	33·0	,, ,,
36	,, ,,	27·7	28·0	,, ,,
37	,, ,,	27·8	30·0	,, ,,
38	,, ,,	27·8	29·0	,, ,,
39	,, ,,	27·5	30·0	,, ,,
40	,, ,,	27·2	31·0	,, ,,
41	,, ,,	27·5	32·0	,, ,,
42	,, ,,	27·8	34·0	,, ,,
43	,, ,,	30·0	27·0	,, ,,
44	,, ,,	29·3	27·0	,, ,,
45	,, ,,	27·1	26·0	,, ,,
46	,, ,,	27·3	29·0	,, ,,
47	,, ,,	26·7	30·0	,, ,,
48	,, ,,	26·6	27·0	,, ,,
49	10″ × 5″ @ 33 lbs.	27·4	30·0	,, ,,
50	,, ,,	27·3	29·0	,, ,,
51	10″ × 5″ @ 29 lbs.	30·5	22·0	,, ,,
52	,, ,,	30·2	24·0	,, ,,
53	,, ,,	29·8	27·0	,, ,,
54	9″ × 7″ @ 58 lbs.	28·2	28·0	,, ,,
55	,, ,,	27·8	28·0	,, ,,
56	,, ,,	26·2	34·0	,, ,,
57	8″ × 5″ @ 30 lbs.	29·5	26·0	,, ,,
58	8″ × 4″ @ 20 lbs.	29·1	24·0	,, ,,
59	7″ × 3¾″ @ 16 lbs.	27·0	23·0	,, ,,
60	,, ,,	28·2	22·0	,, ,,
61	6″ × 5″ @ 25 lbs.	29·1	29·0	,, ,,

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel rolled joists.			
62	6″ × 3″ @ 13 lbs.	30·0	23·0	Bending tests satisfactory
63	” ”	29·2	27·0	” ”
64	” ”	29·8	30·0	” ”
65	” ”	30·8	29·0	” ”
66	” ”	28·7	22·0	” ”
67	” ”	28·4	26·0	” ”
68	5″ × $4\frac{1}{2}$″ @ 18 lbs.	31·9	24·0	” ”
69	” ”	28·6	22·0	” ”
70	5″ × 3″ @ 15 lbs.	30·0	26·0	” ”
71	” ”	29·9	23·5	” ”
72	5″ × 3″ @ 10 lbs.	29·5	23·0	” ”

TABLE No. 9.

Tests on Mild Steel Zed-angles.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel zeds.			
1	8″ × $3\frac{1}{2}$″ × $3\frac{1}{2}$″ × $\frac{1}{2}$″	29·5	21·0	Bending tests satisfactory
2	” ”	30·0	22·0	” ”
3	” ”	28·8	23·0	” ”
4	” ”	27·8	23·0	” ”
5	” ”	29·5	22·0	” ”
6	” ”	29·6	23·0	” ”
7	” ”	29·5	22·0	” ”
8	” ”	29·8	22·0	” ”
9	6″ × $3\frac{1}{2}$″ × 3″ × $\frac{3}{8}$″	30·0	20·0	” ”
10	” ”	29·6	23·0	” ”
11	” ”	28·9	21·0	” ”

TABLE No. 10.

Tests on Mild Steel Trough Flooring.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel trough flooring plates.			
1	**Specimens representing the material**	28·6	27·0	**Bending tests satisfactory**
2	” ”	28·8	24·0	” ”
3	” ”	28·3	28·0	” ”
4	” ”	28·7	26·0	” ”
5	” ”	30·0	29·0	” ”
6	” ”	29·2	27·0	” ”
7	” ”	28·9	31·0	” ”
8	” ”	28·7	28·0	” ”
9	” ”	28·8	30·0	” ”
10	” ”	28·7	26·0	” ”
11	” ”	28·4	24·0	” ”
12	” ”	28·5	27·0	” ”
13	” ”	30·1	27·0	” ”
14	” ”	29·6	25·0	” ”

TABLE No. 11.

Tests on Mild Steel Round Bars.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel round bars.			
1	$6\frac{1}{8}''$ **diameter**	29·2	24·0	**Bending tests satisfactory**
2	$5\frac{1}{2}''$ ”	29·4	22·0	” ”
3	$5\frac{1}{4}''$ ”	28·4	26·0	” ”
4	” ”	27·3	26·0	” ”
5	” ”	28·0	25·0	” ”
6	” ”	29·7	24·0	” ”
7	” ”	28·4	26·0	” ”
8	” ”	27·3	26·0	” ”
9	” ”	28·0	25·0	” ”
10	” ”	29·7	24·0	” ”

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel round bars.			
11	$4\frac{3}{4}''$ diameter	28·4	26·0	Bending tests satisfactory
12	$4\frac{1}{4}''$ "	27·6	30·0	" "
13	" "	29·1	26·0	" "
14	" "	29·5	25·0	" "
15	" "	27·5	27·0	" "
16	" "	27·2	36·0	" "
17	" "	27·6	30·0	" "
18	$4''$ "	30·0	23·0	" "
19	$3\frac{3}{4}''$ "	30·3	26·0	" "
20	" "	28·4	24·0	" "
21	" "	28·8	25·0	" "
22	$3\frac{1}{4}''$ "	28·4	28·0	" "
23	$3\frac{1}{8}''$ "	28·2	21·0	" "
24	$3''$ "	28·1	24·0	" "
25	" "	31·0	15·0	" "
26	" "	28·2	22·0	Further tests from the bars from which No. 25 was taken
27	" "	28·6	23·0	
28	" "	27·8	21·5	
29	" "	29·3	24·0	Bending tests satisfactory
30	" "	29·1	25·0	" "
31	" "	30·0	27·0	" "
32	" "	29·8	28·0	" "
33	$3''$ "	28·6	26·0	" "
34	" "	29·3	27·0	" "
35	" "	29·2	28·0	" "
36	" "	30·0	26·0	" "
37	" "	29·1	26·0	" "
38	" "	30·0	25·0	" "
39	" "	29·7	28·0	" "
40	" "	29·3	21·0	" "
41	" "	28·8	27·0	" "
42	" "	30·0	25·0	" "
43	" "	28·7	27·0	" "
44	$2\frac{3}{4}''$ "	28·8	30·0	" "
45	" "	29·1	31·0	" "
46	" "	29·3	28·0	" "
47	" "	29·0	26·0	" "
48	" "	29·2	27·0	" "
49	" "	28·9	28·0	" "
50	$2\frac{5}{8}''$ "	29·4	31·0	" "
51	$2\frac{1}{2}''$ "	28·2	24·0	" "
52	" "	27·9	25·5	" "
53	" "	29·0	25·0	" "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel round bars.			
54	$2\frac{1}{2}''$ diameter	27·9	24·0	Bending tests satisfactory
55	" "	28·1	25·0	" "
56	" "	28·9	26·0	Tests Nos. 51 to 65 inclusive are the tests for the tie rods shown in Figs. 369–372. See also test No. 67.
57	" "	28·5	22·0	
58	" "	30·5	22·0	
59	" "	28·4	25·0	
60	" "	28·4	25·0	
61	" "	27·3	23·0	Bending tests satisfactory
62	" "	27·0	23·0	" "
63	" "	26·6	32·0	" "
64	" "	26·5	31·0	" "
65	" "	27·2	32·0	" "
66	$2\frac{1}{4}''$ "	29·8	26·0	" "
67	$2''$ "	29·7	27·0	" "
68	$1\frac{3}{4}''$ "	29·0	26·0	" "
69	" "	29·8	26·0	" "
70	" "	29·7	27·0	" "
71	" "	29·5	27·0	" "
72	" "	28·4	26·0	" "
73	$1\frac{1}{2}''$ "	29·3	23·0	" "
74	$1\frac{3}{8}''$ "	28·8	23·0	" "
75	" "	30·0	24·0	" "
76	$1\frac{1}{4}''$ "	27·9	31·0	" "
77	$\frac{7}{8}''$ "	26·8	28·0	" "
78	" "	27·1	32·0	" "

TABLE No. 12.

Tests on Mild Steel Rectangular Bars.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel rectangular bars.			
1	$7'' \times 4''$ rectangular	28·0	29·0	Bending tests satisfactory
2	" "	28·2	31·0	" "
3	" "	28·3	30·0	" "
4	" "	28·8	31·0	" "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel rectangular bars.			
5	7″ × 4″ rectangular	28·3	32·0	Bending tests satisfactory
6	6″ × 4″ „	28·7	28·0	„ „
7	„ „	27·7	28·0	„ „
8	4½″ square	27·9	28·0	„ „
9	4″ square	30·0	28·0	„ „
10	„	27·7	24·0	„ „
11	„	27·0	20·0	„ „
12	5″ × 2⅛″	27·7	31·0	„ „
13	3¾″ square	29·7	26·0	„ „
14	3½″ square	28·4	22·5	„ „
15	„	27·9	23·0	„ „
16	„	27·0	22·5	„ „
17	„	29·8	27·0	„ „
18	3″ square	29·7	26·0	„ „
19	2¾″ square	29·4	24·0	„ „
20	„	29·6	22·0	„ „
21	2½″ square	29·5	27·0	„ „
22	3″ × 1¼″	28·0	22·0	„ „
23	2¼″ square	29·5	26·0	„ „
24	„	29·8	25·0	„ „
25	2″ square	27·5	27·0	„ „
26	1½″ square	26·4	30·0	„ „
27	3½″ × 1¼″	28·0	27·0	„ „

TABLE No. 13.

TESTS ON MILD STEEL PLATES.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel plates, tested lengthways of the plate.			
1	¾″ thick	27·1	29·0	Bending tests satisfactory
2	„	27·1	34·0	„ „
3	„	27·6	29·0	„ „
4	„	27·5	29·0	„ „
5	„	26·9	27·0	„ „

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel plates, tested lengthways of the plate.			
6	¾″ thick	28·0	28·0	Bending tests satisfactory
7	"	28·3	31·0	" "
8	⅝″ thick	29·3	27·0	" "
9	"	28·5	27·0	" "
10	"	27·4	26·0	" "
11	"	28·3	29·0	" "
12	"	28·0	29·0	" "
13	"	29·2	29·0	" "
14	"	28·1	33·0	" "
15	"	26·6	32·0	" "
16	"	27·8	31·0	" "
17	"	28·7	27·0	" "
18	"	28·1	30·0	" "
19	"	27·4	26·0	" "
20	"	27·7	30·0	" "
21	½″ thick	29·5	26·0	" "
22	"	26·9	26·0	" "
23	"	28·0	27·0	" "
24	"	29·1	27·0	" "
25	"	28·3	27·0	" "
26	"	29·1	29·0	" "
27	"	28·7	28·0	" "
28	"	29·6	25·0	" "
29	"	27·8	32·0	" "
30	"	27·7	26·0	" "
31	Chequer or ribbed plates for flooring ½″ thick	28·6	31·0	" "
32	"	28·4	27·0	" "
33	"	30·7	26·0	" "
34	"	29·4	30·0	" "
35	"	29·1	26·0	" "
36	"	28·1	26·0	" "
37	"	29·0	28·0	" "
38	⅜″ thick	28·2	27·0	" "
39	"	29·4	26·0	" "
40	"	26·6	31·0	" "
41	"	28·6	28·0	" "
42	"	28·2	23·0	" "
43	"	28·5	27·0	" "
44	"	28·3	26·0	" "
45	"	28·4	24·0	" "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel plates, tested crossways of the plate.			Specimens Nos. 46 to 78 are from similar plates to those represented by Nos. 1 to 30.
46	$\frac{3}{4}''$ thick	28·0	25·0	Bending tests satisfactory
47	„	28·1	27·0	„ „
48	„	27·8	29·0	„ „
49	„	27·0	30·0	„ „
50	„	27·9	24·0	„ „
51	$\frac{5}{8}''$ thick	28·7	26·0	„ „
52	„	28·2	26·0	„ „
53	„	28·7	26·0	„ „
54	„	28·3	26·0	„ „
55	„	28·6	29·0	„ „
56	„	28·0	29·0	„ „
57	„	26·7	25·0	„ „
58	„	27·3	28·0	„ „
59	„	28·0	26·0	„ „
60	„	27·3	25·0	„ „
61	„	28·2	26·0	„ „
62	$\frac{1}{2}''$ thick	27·0	20·0	„ „
63	„	27·4	28·0	„ „
64	„	29·1	27·0	„ „
65	„	29·3	24·0	„ „
66	„	29·5	25·0	„ „
67	„	27·8	26·0	„ „
68	„	28·2	27·0	„ „
69	„	29·3	24·0	„ „
70	„	26·2	26·0	„ „
71	„	27·7	26·0	„ „
72	„	28·3	29·0	„ „
73	„	29·3	24·0	„ „
74	„	26·2	26·0	„ „
75	„	27·7	26·0	„ „
76	„	28·3	29·0	„ „
77	„	27·8	27·0	„ „
78	„	29·0	25·0	„ „
79	$\frac{3}{8}''$ thick	28·5	25·0	„ „
80	„	30·1	27·0	„ „
81	„	27·2	26·0	„ „
82	„	28·6	25·0	„ „
83	„	27·9	25·0	„ „
84	„	29·1	26·0	„ „
85	„	29·2	22·0	„ „
86	„	27·9	22·0	„ „

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild steel plates, tested crossways of the plate.			
87	$\frac{3}{8}''$ thick	28·5	25·0	Bending tests satisfactory
88	"	28·8	26·0	" "
89	"	28·5	25·0	" "
90	"	30·1	27·0	" "
91	$\frac{5}{16}''$ thick	28·4	24·0	" "
92	"	28·9	25·0	" "
93	"	28·4	26·0	" "
94	"	28·7	25·0	" "
	The following tests represent mild steel plates of various thicknesses, from $\frac{3}{8}''$ to $\frac{3}{4}''$, tested lengthways and crossways			
95	Mild steel plates	28·4	25·0	Tested lengthways
96	" "	28·7	23·0	" crossways
97	" "	29·4	24·0	" lengthways
98	" "	29·6	25·0	" crossways
99	" "	27·8	28·0	" lengthways
100	" "	28·0	27·0	" crossways
101	" "	29·1	28·0	" lengthways
102	" "	29·2	27·0	" crossways
103	" "	29·8	30·0	" lengthways
104	" "	29·8	26·0	" crossways
105	" "	28·8	29·0	" lengthways
106	" "	29·1	25·0	" crossways
107	" "	28·6	26·0	" lengthways
108	" "	28·6	27·0	" crossways
109	" "	29·1	24·0	" lengthways
110	" "	29·7	23·0	" crossways
111	" "	29·0	28·0	" lengthways
112	" "	29·2	27·0	" crossways
113	" "	29·0	26·0	" lengthways
114	" "	29·9	27·0	" crossways
115	" "	28·6	30·0	" lengthways
116	" "	28·9	26·0	" crossways
117	" "	28·8	28·0	" lengthways
118	" "	28·7	25·0	" crossways
119	" "	29·3	27·0	" lengthways
120	" "	29·3	26·0	" crossways

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
121	Mild steel plates.	29·4	25·0	Tested lengthways
122	„ „	29·0	25·0	„ crossways
123	„ „	28·6	26·0	„ lengthways
124	„ „	28·7	24·0	„ crossways
125	„ „	27·8	26·0	„ lengthways
126	„ „	27·8	24·0	„ crossways
127	„ „	29·8	25·0	„ lengthways
128	„ „	29·3	24·0	„ crossways
129	„ „	28·3	26·0	„ lengthways
130	„ „	28·9	27·0	„ crossways
131	„ „	28·6	28·0	„ lengthways
132	„ „	28·6	25·0	„ crossways
133	„ „	28·8	26·0	„ lengthways
134	„ „	28·7	22·0	„ crossways
135	„ „	29·1	28·0	„ lengthways
136	„ „	29·3	23·0	„ crossways
137	„ „	28·2	28·0	„ lengthways
138	„ „	28·6	24·0	„ crossways
139	„ „	29·5	26·0	„ lengthways
140	„ „	29·3	27·0	„ crossways
141	„ „	29·4	25·0	„ lengthways
142	„ „	29·3	23·0	„ crossways
143	„ „	27·8	27·0	„ lengthways
144	„ „	28·0	28·0	„ crossways
145	„ „	28·2	30·0	„ lengthways
146	„ „	27·7	23·0	„ crossways
147	„ „	28·7	29·0	„ lengthways
148	„ „	29·1	28·0	„ crossways
149	„ „	29·0	28·0	„ lengthways
150	„ „	29·1	27·0	„ crossways
151	„ „	28·8	28·0	„ lengthways
152	„ „	29·1	24·0	„ crossways
153	„ „	29·0	26·0	„ lengthways
154	„ „	29·0	25·0	„ crossways
155	„ „	28·4	26·0	„ lengthways
156	„ „	28·8	23·0	„ crossways
157	„ „	29·7	30·0	„ lengthways
158	„ „	29·4	24·0	„ crossways
159	„ „	29·4	25·0	„ lengthways
160	„ „	29·5	26·0	„ crossways
161	„ „	29·2	26·0	„ lengthways
162	„ „	29·5	27·0	„ crossways
163	„ „	28·6	25·0	„ lengthways
164	„ „	28·8	23·0	„ crossways
165	„ „	29·4	25·0	„ lengthways

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
166	Mild steel plates.	29·4	25·0	Tested crossways
167	" "	29·5	23·0	" lengthways
168	" "	29·6	25·0	" crossways
169	" "	28·3	30·0	" lengthways
170	" "	28·6	28·0	" crossways
171	" "	28·9	27·0	" lengthways
172	" "	28·8	24·0	" crossways
173	" "	29·4	26·0	" lengthways
174	" "	29·2	26·0	" crossways
175	" "	28·3	29·0	" lengthways
176	" "	28·4	28·0	" crossways
177	" "	29·0	25·0	" lengthways
178	" "	29·1	27·0	" crossways
179	" "	29·2	26·0	" lengthways
180	" "	29·2	25·0	" crossways

TABLE No. 14.

Tests on Mild Rivet Steel.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild rivet steel.			
1	$\frac{7}{8}''$ diameter	26·4	30·0	Bending tests satisfactory
2	"	27·5	27·0	" "
3	"	26·1	27·3	" "
4	"	26·1	28·4	For the chemical analysis of Test No. 4, see p. 52
5	"	26·3	28·7	Bending tests satisfactory
6	"	27·2	30·0	" "
7	"	27·0	31·0	" "
8	"	27·7	28·0	" "
9	"	27·8	28·0	" "
10	"	27·2	21·0	" "
11	"	28·2	29·8	" "
12	"	28·9	28·5	" "
13	"	28·4	28·9	" "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
	Mild rivet steel.			The tensile strength of
				this rivet steel is slightly
14	$\frac{7}{8}''$ and $\frac{3}{4}''$ diameter	25·1	33·5	below the specified mini-
15	"	25·6	31·0	mum, but in view of the
16	"	24·8	33·0	good elongation and the
17	"	26·2	34·7	satisfactory shop cold
				and temper tests the
				rivets were accepted
18	"	29·6	31·0	Bending tests satisfactory
19	"	29·5	31·0	" "
20	$\frac{3}{4}''$ diameter	25·8	27·5	" "
21	"	27·4	35·0	" "
22	"	28·2	30·0	" "
23	$\frac{5}{8}''$ diameter	27·3	27·0	" "
24	"	28·0	26·0	" "
				The tensile strength of
				Tests Nos. 25 to 29 is
				somewhat high, but the
25	"	31·2	25·5	elongation is good, and
26	"	31·3	27·0	the large number of
27	"	31·5	25·5	mechanical tests (bend-
28	"	32·5	25·0	ing, flattening down,
29	"	31·6	25·8	etc.) made, showed the
				metal to be of high class
				quality, and the rivets
				were accepted
30	Pan-head rivets	26·6	25·0	Bending tests satisfactory
31	Snap-head rivets	27·8	20·0	" "
32	"	29·9	24·0	" "
33	"	30·0	23·0	" "
34	"	24·6	25·0	" "
35	Steel rivets (various)	25·3	31·0	
36	" "	25·1	32·0	Flattening and bending
37	" "	26·4	30·0	(hot and cold) tests gave
38	" "	25·7	31·0	satisfactory results
39	Steel rivets $\frac{31}{32}''$ diam.	29·6	28·0	Bending tests satisfactory
40	" "	30·0	32·0	" "
41	" "	29·8	31·0	" "
42	" $\frac{27}{32}''$ diam.	29·6	32·0	" "
43	" "	30·0	32·0	" "
44	" "	29·8	27·0	" "
45	" "	29·2	34·0	" "
46	" $\frac{23}{32}''$ diam.	28·7	31·0	" "
47	" "	29·2	32·0	" "
48	" "	28·0	32·0	" "

TABLE No. 15.

Tests on Mild Steel for Bolts and Nuts.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate elongation in 8 inches. Per cent.	Remarks.
1	Bolt and nut steel.	29·4	25·0	Bending tests satisfactory
2	" "	29·1	30·0	" "
3	" "	28·4	32·0	" "
4	" "	28·9	28·0	" "
5	" "	27·0	31·0	" "
6	" "	29·1	30·0	" "
7	" "	26·6	26·0	" "
8	" "	29·4	25·0	" "
9	" "	29·1	30·0	" "
10	" "	26·6	26·0	" "
11	" "	26·9	30·0	" "
12	" "	27·3	28·0	" "
13	" "	25·7	29·0	" "
14	" "	26·1	27·0	" "
15	" "	29·9	27·0	" "
16	" "	27·3	29·0	" "
17	" "	27·7	31·0	" "
18	" "	28·3	31·0	" "

TABLE No 16.

Tests on Mild Steel Forgings and other Special Steels.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate extension. Per cent.	Remarks.
	Mild steel forgings.			
1	Shackles	30·6	35·0	Extension on 2 inches
2	"	31·4	37·0	" " "
3	"	31·7	22·5	" 8 "
4	"	31·6	29·0	" 4 "
5	"	31·6	20·0	" 8 "
6	Lifting bolts	30·8	36·0	" 2 "
7	" "	29·3	23·0	" 4 "

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate extension. Per cent.	Remarks.
	Mild steel forgings.			
8	Crossheads	28·8	41·0	Extension on 2 inches
9	"	28·9	23·5	" 8 "
10	Lifting beams	31·6	37·0	" 2 "
11	" "	30·9	37·0	" " "
12	" "	31·5	23·5	" 4 "
13	Crane post	26·7	23·5	" 8 "
14	" "	29·5	26·0	" " "
15	" "	25·4	29·6	" " "
16	Connecting rod	27·9	32·0	" 2 "
17	Sprocket wheel spindles	26·3	35·0	" 3 " For details of this wheel, see Figs. 413–416.
18	Blooms $8\frac{1}{2}''$ square × 10′ 0″ and 7″ square × 14′ 6″	27·8	26·0	Extension on 8 inches
19	Engine forgings	28·4	26·0	" " "
20	" "	28·4	26·0	" " "
21	" "	28·5	28·0	" " "
22	" "	28·5	30·0	" " "
23	" "	28·3	27·0	" " "
24	" "	28·4	28·0	" " "
25	" "	29·9	25·0	" " "
26	" "	28·9	26·0	" " "
27	" "	27·2	28·0	" " "
28	" "	30·0	25·0	" " "
29	" "	27·3	28·0	" " "
30	" "	27·9	27·0	" " "
31	Piston-rod forgings	28·5	24·0	" " "
32	" "	27·4	26·0	" " "
33	" "	28·5	25·0	" " "
34	Shaft forging	27·6	28·0	" " "
35	Forged pins	26·2	29·0	" " "
36	" "	26·3	29·0	" " "
37	Steel tyres and axles	36·0	30·0	Bending tests cold and tempered satisfactory
38	" "	36·5	28·0	" " "
39	" "	39·0	28·0	" " "
40	" "	38·0	29·0	" " "
41	" "	39·6	27·0	" " "
42	" "	40·0	25·0	" " "
43	Steel tyres	44·2	26·0	Extension taken on 3″ for tests 37 to 42 inclusive, and on 2″ for tests 43 to 52 inclusive
44	"	44·1	20·0	
45	Steel crane posts	30·6	35·0	Bending tests satisfactory

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Ultimate extension. Per cent.	Remarks.
	Mild steel forgings.			
46	Steel crane posts	32·2	29·0	Bending tests satisfactory
47	" "	32·8	29·0	" "
48	" "	31·8	33·0	" "
49	" "	32·8	32·0	" "
50	" "	32·2	35·0	" "
51	" "	36·8	20·0	" "
52	" "	33·4	31·0	" " No. 52 is a re-test of No. 51
53	Spring steel 3″ × $\frac{3}{8}$″	45·1	21·0	Not considered satisfactory.
54	" "	52·9	13·0	Replace of No. 53. Extension taken on 6″ for Nos. 53, 54

NOTE.—Tests Nos. 1 to 12 represent material used in the construction of plant for marine works, such as Titans or Goliahs, for the handling of large concrete blocks, etc.

TABLE No. 17.

TESTS ON MILD STEEL FOR SPECIAL PURPOSES.

No. of test.	Description of material.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Remarks.
	Mild steel for special purposes.			
1	Hot drawn weldless steel hexagonal couplings for tie bolts (screwed right and left-handed)	26·42	22·0	For the details of these couplings, see Figs. Nos. 369, 370.
2		26·27	24·5	
3		26·00	23·0	
4		27·05	19·5	
	Steel bars for nuts for bolts.			
5	2″ diameter	28·9	28·0	
6	$2\frac{3}{8}$″ "	29·9	28·0	
7	$2\frac{1}{2}$″ "	26·2	24·0	

No. of test.	Description of materials.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Remarks.
	Steel flats for links for hauling chains.			
8	$6\frac{1}{2}'' \times \frac{7}{8}'' \times 12' 1''$	31·2	23·0	Bending tests satisfactory. This steel was specified to have an ultimate tensile resistance of not less than 30 or more than 33 tons per square inch, with 20 per cent. extension in 8 inches. For the details of the links of the chain made from this steel, see Figs. 410, 411, 422, 423. See also Chemical Analysis, No. 19, p. 54.
9	,, ,,	31·5	23·0	
10	,, ,,	31·9	24·0	
11	,, ,,	31·5	24·0	
12	,, ,,	31·1	25·0	
13	,, ,,	31·2	22·0	
14	,, ,,	32·7	21·0	
15	,, ,,	32·5	22·0	
16	,, ,,	31·0	24·0	
17	,, ,,	31·0	23·0	
18	,, ,,	31·0	24·0	
19	,, ,,	31·2	22·0	
20	,, ,,	31·9	20·0	
21	,, ,,	32·2	24·0	
22	,, ,,	31·9	26·0	
23	,, ,,	31·9	22·0	
24	Flanged nuts for chains	28·9	28·0	Stamped out under the hammer. See Figs. 411, 423.
25	,, ,,	27·0	31·0	
26	Flanged nuts for chains	30·4	31·0	Bending tests satisfactory
27	,, ,,	30·4	30·0	,, ,,
28	,, ,,	28·9	28·0	,, ,,
29	Lewis bolts $\frac{7}{8}''$	29·1	30·0	,, ,,
30	,, ,,	26·6	26·0	,, ,,
31	,, $1\frac{1}{4}''$	28·4	32·0	,, ,,
32	Cotter bolts 2"	29·1	30·0	,, ,,

It is apparent from a consideration of the foregoing results, and from numberless experiments of a similar class which might be quoted, that the present-day processes of steel-making have resulted, especially so far as open-hearth steel is concerned, in a material remarkable for its uniformity of quality, its high tensile resistance, and for the excellent elongation shown by the tests. The highest quality of wrought-iron bar now obtainable may perhaps compare with mild steel favourably as regards ultimate extension, when it has been obtained from the best makers, but will be found to fall considerably below steel in ultimate tensile resistance, while inferior brands of wrought iron can offer no such comparison.

An instance of the great advance shown by the newer material may be found in the tests of plates. In the old days the tensile resistance of wrought-iron plates across the grain was considerably below that lengthways of the grain, while the ultimate extension crossways frequently did not exceed 5 per cent.; a fact which had to be carefully borne in mind in the use of wrought-iron plate across the grain. The results of the tests on mild steel plates are given in the foregoing table, No. 13. It will there be seen that the average ultimate tensile strength of plates tested lengthways, Tests Nos. 1 to 45, is 28·07 tons per square inch, while the same resistance of plates tested crossways, Tests Nos. 46 to 94, is 28·03 tons per square inch.

The average ultimate extension lengthways is 28·5 per cent. in 8 inches, and crossways is 26·23 per cent., showing a slight difference in extension in favour of plates tested lengthways.

Tests Nos. 95 to 180 are the results on plates tested lengthways and crossways under a different contract, but a comparison of the averages gives a similar result to the foregoing series.

The average ultimate tensile strength tested lengthways is 28·82 tons per square inch, and tested crossways is 28·93. The average ultimate elongation in 8 inches, tested lengthways, is 27·00 per cent., tested crossways is 25·28.

A third series of fifty-eight experiments, representing some 200 tons of plates from $\frac{3}{8}$ inch to $\frac{3}{4}$ inch thick, give results in accordance with the foregoing. The ultimate tensile resistance lengthways is 28·24 tons per square inch, with an ultimate extension in 8 inches of 28·6 per cent. Tested crossways, the results are 28·05 tons per square inch and 26·7 per cent. extension.

All three series exhibit a practical uniformity in ultimate tensile strength, whether tested lengthways or crossways, and a difference of from 7 to 9 per cent. in the ultimate extension in favour of the lengthways tests.

Chemical Analysis.—Reference has already been made to the relation between the chemical constitution and the mechanical properties of mild steel, the difficulties surrounding the problem of ascertaining exactly what that relationship is, and of estimating the ultimate strength of a given sample from an analysis of its chemical constituents. The resulting formula, arrived at after an exhaustive examination of the subject by an eminent American authority, has also been given on p. 12.

The following examples of chemical analyses are given, not by

way of elucidating the formulæ above referred to (they are, for this purpose, insufficient in number), but as practical examples of the form of analysis met with in ordinary work, and were made on the same constructive material, the mechanical tests of which have been given in the preceding tables. The chemical analysis and the mechanical test were in all cases made from the same test piece.

The variations to be observed are those usually found in practice.

1. Chemical analysis of sample representing $\frac{5}{8}''$ mild steel plate—

	Per cent.
Carbon	0·200
Silicon	0·019
Sulphur	0·064
Phosphorus	0·046
Manganese	0·565
Iron (by difference)	99·106
	100·000

Tensile test of the above sample—
Ultimate strength, 28·1 tons per square inch.
Elongation, 30·0 per cent. in 8 inches.
Bending tests satisfactory.

2. Chemical analysis of sample representing $\frac{1}{2}''$ mild steel plates—

	Per cent.
Carbon	0·178
Silicon	0·014
Sulphur	0·074
Phosphorus	0·049
Manganese	0·497
Iron (by difference)	99·188
	100·000

Tensile tests of the above sample—
Ultimate strength, 27·5 tons per square inch.
" " 28·4 " "
Elongation, 25·0 per cent. in 8 inches.
" 26·0 " "
Mean ultimate strength, 27·9 tons per square inch.
Mean elongation, 25·5 per cent. in 8 inches.
Bending tests satisfactory.

3. Chemical analysis of sample representing $\frac{3}{8}''$ mild steel plate—

	Per cent.
Carbon	0·220
Silicon	0·014
Sulphur	0·069
Phosphorus	0·050
Manganese	0·497
Iron (by difference)	99·150
	100·000

Tensile test of the above sample—
Ultimate strength, 26·6 tons per square inch.
Elongation, 31·0 per cent. in 8 inches.
Bending tests satisfactory.

4. Chemical analysis of sample representing $\frac{1}{2}''$ mild steel plate—

	Per cent.
Carbon	0·178
Silicon	0·014
Sulphur	0·060
Phosphorus	0·056
Manganese	0·450
Iron (by difference)	99·242
	100·000

Tensile test of the above sample—
Ultimate strength, 27·7 tons per square inch.
Elongation, 28·0 per cent. in 8 inches.
Bending tests satisfactory.

5. Chemical analysis of sample representing mild steel plate—

	Per cent.
Carbon	0·154
Silicon	0·011
Sulphur	0·084
Phosphorus	0·059
Manganese	0·454
Iron (by difference)	99·238
	100·000

Tensile test of the above sample—
Ultimate strength, 29·8 tons per square inch.
Elongation, 27·0 per cent. in 8 inches.
Bending tests satisfactory.

6. Chemical analysis of sample representing mild steel flat—

	Per cent.
Carbon	0·201
Silicon	0·003
Sulphur	0·035
Phosphorus	0·035
Manganese	0·650
Iron (by difference)	99·076
	100·000

Tensile test of the above sample—
Ultimate strength, 28·4 tons per square inch.
Elongation, 31·0 per cent. in 8 inches.
Bending tests satisfactory.

7. Chemical analysis of sample representing 12″ × ⅜″ mild steel flat—

	Per cent.
Carbon	0·170
Silicon	0·015
Sulphur	0·079
Phosphorus	0·043
Manganese	0·720
Iron (by difference)	98·973
	100·000

Tensile test of the above sample—
Ultimate strength, 28·7 tons per square inch.
Elongation, 29·5 per cent. in 8 inches.
The amount of sulphur in this sample is somewhat large, but in other respects the composition is normal, and the bending tests were satisfactory.

8. Chemical analysis of sample representing 5″ × 5″ × ¾″ mild steel angle—

	Per cent.
Carbon	0·116
Silicon	0·023
Sulphur	0·034
Phosphorus	0·042
Manganese	0·684
Iron (by difference)	99·101
	100·000

Tensile test of the above sample—
Ultimate strength, 29·1 tons per square inch.
Elongation, 28·9 per cent. in 8 inches.
Bending tests satisfactory.

9. Chemical analysis of sample representing $3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$ mild steel angle—

	Per cent.
Carbon	0·165
Silicon	0·061
Sulphur	0·060
Phosphorus	0·063
Manganese	0·360
Iron (by difference)	99·291
	100·000

Tensile test of the above sample—
Ultimate strength, 29·2 tons per square inch.
Elongation, 24·0 per cent. in 8 inches.
Bending tests satisfactory.

10. Chemical analysis of sample representing $3'' \times 3'' \times \frac{1}{2}''$ mild steel angles—

	Per cent.
Carbon	0·316
Silicon	0·053
Sulphur	0·053
Phosphorus	0·056
Manganese	0·360
Iron (by difference)	99·162
	100·000

Tensile test of the above sample—
Ultimate strength, 28·5 tons per square inch.
Elongation, 25·0 per cent. in 8 inches.
Bending tests satisfactory.

11. Chemical analysis of sample representing $3'' \times 3'' \times \frac{1}{2}'$ steel angles—

	Per cent.
Carbon	0·198
Silicon	0·072
Sulphur	0·047
Phosphorus	0·062
Manganese	0·342
Iron (by difference)	99·279
	100·000

Tensile test of the above sample—
Ultimate strength, 29·2 tons per square inch.
Elongation, 24·0 per cent. in 8 inches.
Bending tests satisfactory.

12. Chemical analysis of sample representing 3″ × 3″ × ½″ steel angles—

	Per cent.
Carbon	0·200
Silicon	0·080
Sulphur	0·069
Phosphorus	0·067
Manganese	0·468
Iron (by difference)	99·116
	100·000

Tensile test of the above sample—
Ultimate strength, 28·7 tons per square inch.
Elongation, 23·5 per cent. in 8 inches.
Bending tests satisfactory.

13. Chemical analysis of sample representing mild steel for rivets—

	Per cent.
Carbon	0·130
Silicon	0·026
Sulphur	0·055
Phosphorus	0·056
Manganese	0·360
Iron (by difference)	99·373
	100·000

Tensile test of the above sample—
Ultimate strength, 26·1 tons per square inch.
Elongation, 28·4 per cent. in 8 inches.

14. Chemical analysis of sample representing 7″ × ½″ flats—

	Per cent.
Carbon	0·174
Silicon	0·009
Sulphur	0·074
Phosphorus	0·038
Manganese	0·547
Iron (by difference)	99·158
	100·000

Tensile test of the above sample—
Ultimate strength, 28·0 tons per square inch.
Elongation, 22·0 per cent. in 8 inches.
Bending tests satisfactory.

15. Chemical analysis of sample representing $2\frac{1}{2}'' + \frac{3}{8}''$ flats—

	Per cent.
Carbon	0·176
Silicon	0·010
Sulphur	0·073
Phosphorus	0·042
Manganese	0·688
Iron (by difference)	99·011
	100·000

Tensile test of the above sample—
Ultimate strength, 28·80 tons per square inch.
Elongation, 31·0 per cent. in 8 inches.
Bending tests satisfactory.

16. Chemical analysis of sample representing $2\frac{1}{2}'' + 2\frac{1}{2}''$ angles—

	Per cent.
Carbon	0·169
Silicon	0·026
Sulphur	0·083
Phosphorus	0·050
Manganese	0·410
Iron (by difference)	99·262
	100·000

Tensile tests of the above sample—
Ultimate strength, 29·3 tons per square inch.
Elongation, 25·0 per cent. in 8 inches.
Ultimate strength, 27·8 tons per square inch.
Elongation, 24·0 per cent. in 8 inches.

17. Chemical analysis of a sample representing $\frac{1}{2}''$ thick mild steel plate—

	Per cent.
Carbon	0·190
Silicon	0·020
Sulphur	0·072
Phosphorus	0·064
Manganese	0·612
Iron (by difference)	99·042
	100·000

Tensile tests of the above sample—
Lengthways, Ultimate strength, 28·6 tons per square inch.
Elongation, 27·0 per cent. in 8 inches.
Crossways, Ultimate strength, 28·8 tons per square inch.
Elongation, 24·0 per cent. in 8 inches.

18. Chemical analysis of a sample representing $\frac{5}{8}''$ thick mild steel plate—

	Per cent.
Carbon	0·154
Silicon	0·009
Sulphur	0·067
Phosphorus	0·057
Manganese	0·468
Iron (by difference)	99·245
	100·000

Tensile tests of the above sample—
Lengthways, 28·9 tons per square inch.
Elongation, 26·0 per cent. in 8 inches.
Crossways, 28·6 tons per square inch.
Elongation, 28·0 per cent. in 8 inches.

19. Chemical analysis of a sample representing $6\frac{1}{2}'' \times \frac{7}{8}''$ flats, used in the manufacture of links for hauling chains—

	Per cent.
Carbon	0·238
Silicon	0·046
Sulphur	0·057
Phosphorus	0·070
Manganese	0·652
Iron (by difference)	98·937
	100·000

Results of tensile tests of the above sample (Analysis No. 19)—

Breaking strain. Tons per square inch.	Elongation in 8 inches. Per cent.
32·2	26·0

This specimen is from the same class of material represented in Tests Nos. 8 to 23 inclusive (Table No. 17), under the heading of "Steel for Special Purposes." It represents a class of steel slightly higher in ultimate tensile strength, and having a slightly lower percentage of elongation than the bulk of the material represented in the tables. The processes through which the steel went in the

manufacture of the finished link (shown in Figs. 410, 422) proved the excellence of the material.

20. Chemical analysis of a sample representing 3″ × 3″ × ⅜″ mild steel angles—

	Per cent.
Carbon	0·166
Silicon	0·023
Sulphur	0·043
Phosphorus	0·053
Manganese	0·587
Iron (by difference)	99·128
	100·000

Tensile tests of the above sample—
Ultimate strength, 28·3 tons per square inch.
Elongation, 25·0 per cent. in 8 inches.
Bending tests satisfactory.

21. Chemical analysis of a sample representing 3½″ × 3½″ × ⅜″ mild steel angles—

	Per cent.
Carbon	0·128
Silicon	0·042
Sulphur	0·053
Phosphorus	0·069
Manganese...	0·635
Iron (by difference)	99·073
	100·000

Tensile tests of the above sample—
Ultimate strength, 29·8 tons per square inch.
Elongation, 27·0 per cent. in 8 inches.
Bending tests satisfactory.

22. Chemical analysis of a sample representing ½″ mild steel plate—

	Per cent.
Carbon	0·206
Silicon	0·038
Sulphur	0·081
Phosphorus	0·043
Manganese...	0·637
Iron (by difference)	98·995
	100·000

Tensile tests of the above sample—
Lengthways, 27·2 tons per square inch
Crossways, 27·8 tons per square inch.
Elongation, Lengthways, 2·70 per cent. in 8 inches.
Crossways, 25·0 per cent. in 8 inches.
Bending tests satisfactory.

For comparison with the foregoing mechanical tests and chemical analyses of mild steel, the following table of the results of mechanical tests on wrought-iron bars, rectangular and round, is appended. The material here indicated represents good present-day practice in the manufacture of wrought iron, and is of as high a quality as could be readily obtained under ordinary commercial conditions. As the manufacture of wrought-iron plates is no longer of the importance that once belonged to it, no comparison between the old and the modern material need in this respect be instituted.

TABLE No. 18.

Results of Physical Tests on Wrought-iron Bars, Rectangular and Round.

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Remarks.
	Wrought-iron bars.			
1	8¼″ square from 6′ 11″ to 7′ 8″ long	22·3	12·0	These bars were specified to have not less than 22 tons ultimate tensile strength per square inch, and 10 per cent. elongation in 8 inches
2	„ „	22·8	17·0	
3	„ „	20·3	8·0	
4	„ „	22·1	27·0	
5	„ „	22·5	24·0	
6	„ „	22·5	27·0	
7	„ „	22·3	26·0	
8	„ „	22·5	27·0	
9	„ „	22·3	26·0	
10	„ „	24·1	21·0	
11	„ „	23·6	22·0	Forge tests were satisfactory
12	„ „	23·5	23·0	„
13	„ „	23·3	23·0	„
14	„ „	23·3	24·5	„
15	„ „	22·3	24·0	„
16	„ „	22·6	27·5	„

No. of test.	Description of section tested.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Remarks.
	Wrought-iron bars.			
17	3½″ square from 6′ 11″ to 7′ 8″ long	22·9	21·5	Forge tests were satisfactory
18	,, ,,	23·2	24·0	,,
	Wrought-iron flats.			
19	8½″ × 2½″	23·0	27·0	
20	4½″ × 1⅞″	22·7	26·0	
21	3½″ × 1½″	23·1	25·0	
	Wrought-iron round bars.			
22	4⅝″ diameter	22·6	29·0	Specified to have not less than 22 tons ultimate tensile strength, and 10 per cent. elongation
23	4⅜″ ,,	22·7	28·0	
24	3⅜″ ,,	23·1	23·0	
25	2″ ,,	23·9	27·0	
26	1½″ ,,	22·8	34·0	
27	,, ,,	23·0	28·5	
28	,, ,,	24·9	16·0	For the chemical analysis from drillings from tests Nos. 28 to 36 inclusive, mixed well together, see below.
29	,, ,,	24·9	20·0	
30	,, ,,	25·4	13·0	
31	1⅛″ ,,	22·9	28·0	
32	,, ,,	23·1	28·0	
33	⅞″ ,,	26·3	15·0	
34	,, ,,	24·1	17·0	
35	¾″ ,,	26·2	26·0	
36	,, ,,	23·9	18·0	

The following chemical analysis of a mixture of the samples whose mechanical tests are described by tests Nos. 28 to 36 inclusive (Table No. 18) is of interest, when compared with the analyses of mild steel given above.

Chemical analysis of a sample representing wrought-iron round bars ¾ inch to 1½ inch diameter—

	Per cent.
Carbon	0·042
Silicon	0·118
Sulphur	0·030
Phosphorus	0·237
Manganese	0·162
Slag	0·878
Iron (by difference)	98·533
	100·000

The results of the tensile tests are given in the Table No. 18, tests Nos. 28 to 36 inclusive, the mean ultimate tensile strength being 24·6 tons per square inch, and the mean ultimate elongation 20·1 per cent. in 8 inches, ranging from 13 to 28 per cent.

The student will observe in the above analysis the low percentage of carbon and manganese, the high percentage of phosphorus, and last, but not least, the presence of slag to the extent of nearly 1 per cent.; the material is, however, of good quality.

The foregoing remarks on the mechanical tests and chemical analyses required for the determination of the quality and strength of structural mild steel could hardly be considered complete, even in the elementary form in which they have been presented, without a reference to that important branch of laboratory investigation, the microscopic examination of the structure of metals. More than a reference to this subject is not, however, within the scope of these notes, the subject demanding the discussion of refinements of methods and of results which are beyond the more rough-and-ready determinations of commercial tests and analyses.

The results to be obtained from microscopic investigation give ample promise of high scientific value in the future, in the determination of the general equations subsisting between the mechanical qualities, chemical constituents, and microscopic structure of the specimens examined, but for the everyday requirements of the designer of structural steel-work, this method has not yet superseded the more ordinary process of mechanical testing and chemical analysis above referred to, especially when the comparatively small area of surface of section examined under even moderately high powers is taken into consideration, while the amount of material to be accepted or rejected is measured in tons.

The following remarks contained in the presidential address of the Iron and Steel Institute for 1901[1] are suggestive as to the possible issue in the near future not only of the competition at present existing between the Bessemer and open-hearth processes, but also of that between the acid and basic subdivision of the latter process:—

"Notwithstanding all that has been done chiefly in the direction of securing larger output and greater regularity in the product, the waste in the Bessemer process remains practically the

[1] Iron and Steel Institute. Presidential address by Mr. William Whitwell, May, 1901.

same as it was in the early days. Although the purposes for which Bessemer steel (acid and basic) is now being used have increased enormously—fully one-half the make in this country being used for other purposes than railway material—it seems probable that by reason of cheaper methods of producing steel, the Bessemer processes will have in future much more serious competition than has been the case in the past. The recent modification of the open-hearth process by Bertrand-Thiel and by Talbot, aided, as they are certain to be, by the labour-saving appliances already in successful operation, seem to indicate that we are now on the verge of effecting still greater economies in our steel-producing methods, and unless some means can be devised of reducing the waste in the Bessemer converters, the Bessemer processes, which have served the world so well in the past, are likely to be superseded.

"For regularity and reliability of product the Siemens acid steel process stands pre-eminent. By far the greater part of the open-hearth steel in this country is made by this process, owing to the facilities for obtaining a cheap and efficient supply of hematite pig-iron, and so long as such conditions continue to exist it will undoubtedly hold its own. But the basic open-hearth process is advancing with rapid strides, and is seriously challenging the position of the acid process as regards the cheapness of its product; and this fact, coupled with what I have already said on the question of the pure ore supply, would seem to point to the conversion of many acid hearths into basic at no very distant date. In the developments of the iron and steel industries in the future the basic Siemens process will no doubt claim most attention."

In connection with the remarks of the authority above quoted, the following comparison of the total outputs of steel in the United Kingdom, Germany, and the United States for the year 1899 will be found of interest:—

The total amount of steel manufactured in Great Britain in the year mentioned was 4,855,325 tons. Of this amount 1,825,074 tons were produced by the Bessemer process and 3,030,251 tons by the open-hearth process. Of the Bessemer process 71·6 per cent. was acid and 28·4 per cent. basic steel. Of the open-hearth process 90·3 per cent. was acid and 9·7 per cent. was basic steel. The Bessemer process produced in 1899 about 37 per cent. of the whole amount manufactured, whereas ten years previously, in 1889, its percentage of the gross output was 60.

In Germany the total production of steel in 1899 amounted to 6,290,434 metric tons. Of this amount 63·2 per cent. was basic converter, 26·9 per cent. basic open-hearth, and 9·9 per cent. was acid Bessemer.

In the United States for the same year the total production of steel was 10,662,170 tons of 2240 lbs. Of this amount 71·7 per cent. was Bessemer, 27·7 per cent. was open-hearth, and 1·2 per cent. crucible and special steel. Of the open-hearth steel 70·6 per cent. was basic and 29·4 per cent. was acid steel. In 1902 the total production of steel in the United States amounted to 14,994,200 tons, of which 5,687,729 tons were open-hearth, or nearly 38 per cent. of the total quantity. Of the open-hearth steel 21 per cent. was acid and 79 per cent. basic.

In 1904, the total production of steel in the United States amounted to 13,859,887 tons, of which 56·6 per cent. was Bessemer, 42·7 per cent. was open-hearth, and 0·7 per cent. was crucible or special steel. Of the open-hearth steel manufactured in the United States in 1904, 86·4 per cent. was basic and 13·6 per cent. was acid. Practically all the Bessemer steel was acid.

In Great Britain, the total production of steel in 1904 was 5,026,879 tons, of which 35·4 per cent. was Bessemer and 64·6 per cent. was open-hearth.

Of the open-hearth steel manufactured in Great Britain in 1904, 79·6 per cent. was acid and 20·4 per cent. was basic.

Cast Steel.—This most valuable material has of late years come into prominent use in a variety of forms, although in ordinary building construction the expense attending its adoption has been, and still is, the principal obstacle to its extended use in that direction. A material possessing some four times the direct tensile resistance of ordinary cast iron, and a range of elasticity to which cast iron offers but little parallel, appears at first sight an ideal metal to be cast into constructional forms. But, as above stated, the high price of steel castings at present forms an effective commercial obstacle to any rivalry with the cheaper metal in ordinary construction. There are also other facts to be taken into account which somewhat discount the advantages of increased strength and elasticity. These are, in the present stage of the art of steel-founding, first, the want of finish on the exterior of the casting, the roughness of the surface contrasting somewhat unfavourably with the smooth finish obtainable on the skin of a good casting in iron—a result commonly supposed to be due to the high temperature

of cast steel, and its injurious effect upon the surface of the mould exposed to the intense heat and attrition of the molten metal. The second defect, of a more vital nature, is one which probably causes the inspectors and users of this material considerably more anxiety than the want of external finish. It consists in the almost invariable presence of gas-holes, or, as would be called in iron castings, air- or blow-holes, more or less numerous, mainly of small dimensions, but frequently not showing themselves on the surface of the casting until machining has revealed their existence, and possibly not even then.

The judgment of the inspector will frequently be exercised as to the extent to which such defects may be considered to affect the soundness of the casting, and an indiscriminate rejection of work on this ground only is generally considered an impracticable course to adopt in the present condition of steel founding.

The problem will again and again present itself to the mind of the inspecting official as to how far a gas-hole manifesting itself on the surface, let us say of a machined casting, may extend into the body of the work, what may be its cubic capacity or its ramifications, and to what extent is it likely to interfere with the practical soundness of the casting, and with its fitness for the purpose for which it has been designed?

When the depth of such a defect cannot be well ascertained by pricking with a wire, owing possibly to its crooked shape, a measure of its cubic capacity may be obtained by pouring water into the hole, and observing the volume of water absorbed.

But when every allowance has been made for the defects above mentioned, it must still be conceded that the high tensile resistance, elasticity, and toughness of this material render it extremely valuable under certain conditions.

But little is at present known as to its compressive resistance in the form of long columns, or in the compression flange of simple girders, and the proportions of girder flanges determined by the early and well-known experiments of Hodgkinson would probably require considerable modification in the use of cast steel for simple H-shaped girders.

In machinery and engine work the use of cast steel is extensive and various. Spur wheels of all sizes up to large diameters are in frequent use, though the high rate of shrinkage of cast steel as compared with cast iron gives some little trouble in this direction occasionally.

Mitre wheels, small gearing and clutches, and generally details exposed to severe shock are all instances of its use.

In ship construction we have perhaps the finest examples of the use of this material in large and complicated castings often of several tons in weight, as in rambow castings, screw frames (known in the shipbuilding yard as "spectacles"), rudder frames, and the like.

In the Table of tests which follows, a considerable variety of purposes to which this material has been applied will be observed. Thus in Table No. 20, tests Nos. 1 to 40, we find a series of tests of steel castings used for pawl racks in slipways for hauling craft up for repairs having a displacement of some 200 tons or more. The racks having to resist the shock of the pawls in the event of the hauling apparatus giving out, were required to be of a tough and very strong material, and cast steel was used for the purpose.

In Table No. 20, tests Nos. 41 to 97, we have a series of tests on cast-steel bollards for mooring purposes on a wharf or breakwater wall.

Tests Nos. 270 to 272 are the results from castings for wheels to bogie waggons carrying concrete blocks of 50 tons weight. These are small wheels of crucible cast steel.

Tests Nos. 98 to 193 are from steel castings for the roller paths of large sliding caissons for dock entrances, these paths being laid under water. Tests Nos. 194 to 226 are from the roller castings themselves.

Specimens from cast-steel cylinders for hydraulic rams, and their covers, together with various other items of machine construction, also find their place in the tables.

In several cases the details of these castings are found in the illustrations referred to, in order that the student may fully realize the class of work to which the tests have reference.

It is perhaps unnecessary to remark that when cast steel is specified, it is desirable to be sure that cast steel is supplied. All is not cast steel that is called by that name.

An examination of tests Nos. 1 to 8 in Table No. 20 will show considerable fallings off in the metal offered from the specified standard, but the improved results successively indicated in tests Nos. 9 and onward show the beneficial result of a steady adherence to the requirements of the specification.

The tests applied to steel castings usually consist of tensile and bending tests on specimens cast on the castings and cut off

for that purpose after annealing. Large castings are frequently hammered, or let fall from a height, to test their soundness.

The annealing of steel castings is frequently adopted, the castings being re-heated in a specially constructed furnace, and allowed to cool gradually for certain specified periods. All the tests enumerated in the following tables are on annealed specimens.

The bending tests are commonly carried out on square or round bars bent cold by an hydraulic press over a curved block, the radius of which is specified, and it is well for the student in drawing up his requirements in this particular to be clear as to the meaning of the clause defining the angle *through* which the bar is to bend. Thus to specify that a bar shall bend *to* an angle of 45°, appears to leave it an open question as to whether it shall bend *through* an angle of 45° or *through* an angle of 135°. The use of the term "interior angle," or, better still, the figured diagram, will, however, leave the requirement intelligible and free from dispute. Considerable variations will be observed in the tables with respect to the angles bent through, as compared with the ultimate elongation in tensile tests, and the causes of this variability are not very obvious. It is possible, however, that they are partly due to differences of conditions in the process of annealing. The following table, No. 19, shows, in general terms, the practice with respect to annealing of steel castings, position of test-pieces, etc., of five of the leading cast-steel manufacturers in this country at the present time.

TABLE No. 19.

SHOWING THE PRACTICE WITH RESPECT TO THE ANNEALING OF STEEL CASTINGS, AND POSITION OF TEST-PIECES, OF FIVE LEADING CAST-STEEL MANUFACTURERS IN GREAT BRITAIN.

Description of furnace.	Siemens open-hearth steel furnaces.	Special Siemens-Martin furnaces.	Siemens-Martin open-hearth furnaces.	Siemens-Martin open-hearth furnaces.	Siemens-Martin open-hearth furnaces.
Annealing furnace.	Secret method of annealing. Castings annealed for at least ten days.	Castings annealed in special furnaces heated by gas for about four to six days.	Castings about seven days in annealing furnace.	No information given.	Castings heated to a bright cherry-red for about three days.
Position of test-pieces on castings and method of annealing them.	These are always attached to the main casting and annealed with them. Their position on the casting is usually kept as near as possible to the runner. The tensile and bending specimens are taken from the same piece.	These are always attached to the main casting and annealed in position. They are not cast in any particular way on the piece. The tensile and bending specimens are taken from the same piece.	These are always attached to the main casting and annealed in position. The test-piece is invariably at the bottom of the casting, the runner being at the top. The tensile and bending specimens are taken from the same piece.	These are always attached to the main casting and annealed in position. The test-pieces are usually placed at the joints of the moulding boxes. The position of the runner varies. The tensile and bending specimens are separate on the casting.	The test-pieces are cast in a vertical position at the bottom of the mould in the case of large castings. They are cast separately with smaller pieces. They are annealed with their castings. The tensile and bending specimens are taken from the same piece.

The great advantage possessed by cast steel over cast iron in the ductility exhibited even by the smallest angle bent through, given in the table, is sufficiently obvious.

Fig. 4 is an elevation of a typical specimen of cast-steel test bar nominally 1 inch square, bent by hydraulic pressure over a block having a radius of 1½ inch, to an interior angle of 70°, and remaining unbroken.

Fig. 5 is a section of the bar on the line *a-b*, showing the

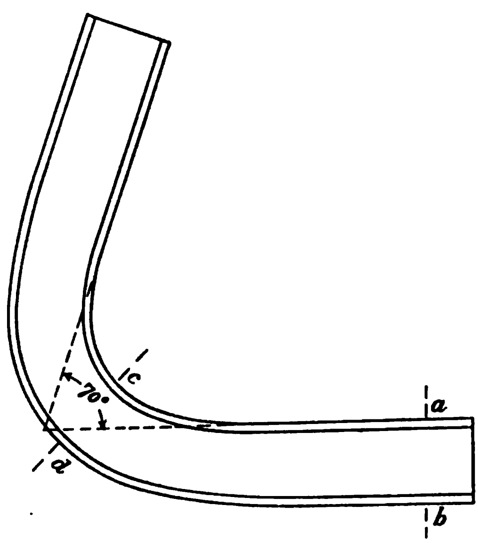


Fig. 4.
Scale, half full size.

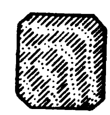


Fig. 5.
Scale, half full size.

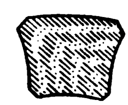


Fig. 6.
Scale, half full size.

section of the bar, with its square edges machined or filed off. (The whole bar was machined.) Fig. 6 gives the section through the bend of the bar on the line *c, d,* and shows the flow of material due to the process, the concave fibres in compression being laterally extended, while the convex surface is contracted, accompanied by a slight hollowness in the sides, the whole figure well exhibiting the ductility of the material.

The tensile tests are made in the usual way on specimens which are sometimes 8 inches in length between the shoulders, but more commonly 2 inches in length, the latter dimension being that usually preferred by the steel founder, as being more advantageous from his point of view.

TABLE No. 20.

Tensile and Bending Tests on Cast-steel Bars.

All bars machined approximately 1 inch square (with the exceptions described) for bending tests, and prepared in the usual way for tensile tests.

Bending tests were made over a block having a radius of 1½ inch, and were bent cold under hydraulic pressure. All specimens, both for bending and tensile tests, were annealed with the castings which they were intended to represent.

The material for the greater part of the castings was supplied under the specification given on p. 19.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
1	Pawl racks	18·7	5·0	...	...	All these results being so much below the specified requirements, the castings represented were rejected
2	"	18·7	3·1			
3	"	19·8	3·7			
4	"	22·5	2·25			
5	"	24·0	4·25			
6	"	24·2	3·75			
7	"	23·3	2·75			
8	"	21·8	2·5			
9	"	32·5	14·75	...	...	Good fracture
10	"	26·75	10·50	...	...	"
11	"	37·50	16·75	...	...	"
12	"	26·90	9·75	...	...	"
13	"	27·50	15·75	...	...	"
14	"	29·75	8·0	...	...	Below the specified elongation
15	"	32·1	18·75	...	...	Good fracture
16	"	32·2	20·5	...	...	"
17	"	33·6	20·0	...	...	"
18	"	33·9	20·75	...	...	"
19	"	33·85	11·0	...	...	"
20	"	32·45	12·0	...	...	"
21	"	31·5	22·2	...	...	"
22	"	31·1	13·5	...	...	"
23	"	31·1	21·25	...	...	"
24	"	30·9	23·25	...	...	"

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
25	Pawl racks	29·1	23·0	...	...	Good fracture
26	"	29·0	23·5	...	...	"
27	"	30·2	19·75	...	...	"
28	"	28·6	19·5	...	...	"
29	"	29·8	23·1	...	...	"
30	"	29·2	19·5	...	...	"
31	"	31·0	23·0	...	...	"
32	"	30·4	24·5	...	...	"
33	"	32·1	21·8	...	...	"
34	"	32·0	20·5	...	...	"
35	"	31·5	19·0	...	...	"
36	"	32·0	21·75	...	...	"
37	"	31·75	24·75	...	...	"
38	"	33·25	21·5	...	...	"
39	"	27·85	...	...	...	Broke outside datum points
40	"	34·15	22·25	...	...	These specimens were cut from the racks themselves, and not from runners cast at the same time as the racks
41[1]	Bollards	31·8	...	26·0[2]	105	
42	"	31·02	...	26·0	95	
43	"	32·8	...	21·0	90	
44	"	31·5	...	21·5	115	
45	"	33·25	...	20·0	110	
46	"	32·9	...	20·5	120	
47	"	31·9	...	15·0	90	
48	"	31·8	...	17·0	88	
49	"	31·55	...	15·0	87	
50	"	32·4	...	16·5	105	
51	"	32·6	...	16·0	100	
52	"	31·85	...	16·0	102	

[1] Test Nos. 41 to 63 inclusive for bending were made on bars $\frac{7}{8}$ inch to $\frac{15}{16}$ inch square. Bending tests Nos. 64 to 97 inclusive were made on 1 inch diameter round bars.

[2] Specimens for testing in tension were from 0·74 to 0·81 inch in diameter, and from 0·48 to 0·51 square inch in area.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
53	Bollards	29·6	...	16·0	110	
54	"	30·4	...	14·0	115	
55	"	29·4	...	18·0	105	
56	"	32·6	...	18·0	105	
57	"	32·75	...	17·0	110	
58	"	31·80	...	17·0	105	
59	"	32·75	...	18·0	95	
60	"	32·90	...	19·0	105	Sound
61	"	31·8	...	17·0	110	"
62	"	31·6	...	18·0	105	Broke
63	"	31·3	...	16·0	100	Sound
64	"	33·5	...	19·0	96	"
65	"	34·6	...	18·5	105	Broke
66	"	33·4	...	14·5	95	"
67	"	34·8	...	18·0	105	Sound
68	"	28·8	...	18·0	100	Broke
69	"	30·0	...	16·0	100	Sound
70	"	28·8	...	19·0	113	Broke
71	"	33·8	...	16·0	100	Sound
72	"	34·3	...	16·0	90	"
73	"	30·9	...	24·0	90	"
74	"	30·6	...	23·0	90	"
75	"	30·5	...	25·0	90	"
76	"	33·3	...	26·0	90	"
77	"	32·0	...	26·0	90	"
78	"	32·9	...	22·0	95	"
79	"	32·2	...	23·0	90	"
80	"	32·4	...	26·0	90	"
81	"	32·2	...	23·0	90	"
82	"	32·5	...	20·0	90	"
83	"	32·0	...	27·0	90	"
84	"	30·8	...	24·0	90	"
85	"	32·3	...	23·0	90	"
86	"	33·0	...	30·0	90	"
87	"	33·8	...	25·0	90	"
88	"	33·0	...	28·0	90	"
89	"	33·7	...	21·0	90	"
90	"	34·2	...	27·0	90	"
91	"	33·9	...	22·0	90	"
92	"	34·2	...	26·0	90	"
93	"	32·2	...	25·0	90	"
94	"	35·2	...	21·0	90	"
95	"	34·2	...	23·0	90	"
96	"	33·1	...	23·5	90	"
97	"	33·2	...	23·0	90	"

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
98	Roller[1] path castings	32·2	...	19·0	45	Sound
99	„	29·6	...	28·0	50	„
100	„	29·0	...	14·0	50	„
101	„	31·3	...	19·0	50	„
102	„	28·9	...	13·0	51	„
103	„	33·5	11·5	...	45	Broke
104	„	31·0	...	28·0	50	Sound
105	„	29·1	12·5	...	50	„
106	„	30·3	...	29·0	50	„
107	„	29·1	...	31·0	60	„
108	„	27·3	...	19·0	60	„
109	„	29·4	12·0	...	70	Broke
110	„	31·3	...	28·0	57	Sound
111	„	33·2	10·5	...	50	Broke
112	„	29·5	20·0	...	75	„
113	„	31·8	21·0	...	70	„
114	„	30·2	...	30·0	50	„
115	„	28·8	26·0	...	48	Sound
116	„	27·0	13·5	...	51	„
117	„	29·4	13·5	...	75	Broke
118	„	32·2	...	16·0	45	Sound
119	„	29·2	25·0	...	50	„
120	„	29·3	15·0	...	55	Broke
121	„	29·5	16·0	...	50	Sound
122	„	33·3	...	18·0	50	Broke
123	„	31·6	15·0	...	40	„
124	„	31·7	...	14·0	35	„
125	„	31·6	14·0	...	80	„
126	„	34·0	13·0	...	70	„
127	„	31·7	...	29·0	125	„
128	„	29·2	...	25·0	100	„
129	„	31·6	14·0	...	125	Sound
130	„	29·9	...	15·0	70	Broke
131	„	29·6	10·0	...	90	„
132	„	30·6	15·0	...	90	Sound
133	„	29·7	12·0	...	135	„
134	„	30·7	...	24·0	145	„
135	„	33·3	13·0	...	65	Broke
136	„	34·2	14·0	...	140	Sound
137	„	29·1	11·0	...	145	„

[1] The details of these roller paths are given in Figs. 403, 404, and a description will be found on p. 410.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
138	Roller path castings	29·9	...	12·0	45	Broke
139	"	33·7	...	16·0	50	"
140	"	29·7	10·0	...	70	"
141	"	30·5	...	30·0	55	"
142	"	30·5	...	15·0	130	"
143	"	29·7	10·0	...	70	"
144	"	31·2	11·0	...	75	"
145	"	28·1	...	28·0	145	"
146	"	31·1	...	15·0	80	"
147	"	29·8	12·0	...	75	"
148	"	30·2	...	20·0	145	Sound
149	"	26·1	...	8·0	85	Broke
150	"	27·8	...	10·0	70	"
151	"	29·6	...	18·0	70	"
152	"	29·2	...	23·0	95	"
153	"	31·5	...	19·0	95	"
154	"	31·2	...	16·0	90	Sound
155	"	29·6	...	10·0	90	Broke
156	"	27·9	17·0	...	100	"
157	"	29·9	...	24·0	75	"
158	"	30·5	...	19·0	55	"
159	"	32·1	...	21·0	60	"
160	"	32·4	...	20·0	105	"
161	"	28·5	...	10·0	45	"
162	"	33·9	...	24·0	45	"
163	"	30·5	...	17·0	105	"
164	"	28·0	...	21·0	70	"
165	"	30·2	...	29·0	52	Sound
166	"	31·7	15·0	...	75	Broke
167	"	31·0	...	16·0	110	Sound
168	"	30·0	13·0	...	95	"
169	"	32·2	12·0	...	90	"
170	"	29·2	19·0	...	70	Broke
171	"	31·5	...	17·0	65	Broke
172	"	27·1	10·0	...	65	"
173	"	28·7	...	15·0	80	Sound
174	"	27·6	13·0	...	60	Broke
175	"	27·7	25·0	...	55	"
176	"	32·9	...	15·0	80	"
177	"	30·2	...	20·0	135	Sound
178	"	31·2	11·0	...	75	Broke
179	"	28·9	11·0	...	62	"
180	"	30·1	13·0	...	52	"

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
181	Roller path castings	28·4	11·0	...	76	Broke
182	"	28·3	12·0	...	67	"
183	"	28·7	10·0	...	122	Sound
184	"	32·2	...	26·0	65	Broke
185	"	28·0	10·0	...	95	"
186	"	31·2	...	25·0	77	"
187	"	30·0	11·0	...	75	"
188	"	32·0	15·0	...	145	Sound
189	"	27·5	18·0	...	40	Broke
190	"	27·6	8·0	...	60	"
191	"	27·9	8·0	...	87	"
192	"	28·8	8·0	...	67	"
193	"	30·3	15·0	...	85	"
194	Roller[1] castings	29·2	...	26·0	50	Sound
195	"	33·5	11·5	...	45	Broke
196	"	28·8	26·0	...	48	Sound
197	"	29·5	16·0	...	50	"
198	"	28·7	...	34·0	49	"
199	"	28·7	34·0	...	64	Broke
200	"	30·5	14·5	...	135	Sound
201	"	29·5	18·5	...	65	Broke
202	"	29·6	13·5	...	85	"
203	"	31·0	14·5	...	60	"
204	"	25·4	12·0	...	75	"
205	"	27·3	10·0	...	80	"
206	"	27·9	12·5	...	75	"
207	"	28·3	...	19·0	150	Sound
208	"	25·4	13·5	...	80	Broke
209	"	29·4	13·5	...	115	"
210	"	29·6	19·0	...	95	"
211	"	29·7	16·0	...	110	"
212	"	28·0	13·5	...	60	"
213	"	29·3	13·5	...	70	"
214	"	31·1	14·0	...	105	Sound
215	"	28·0	20·0	...	90	Broke
216	"	29·2	...	24·0	45	"
217	"	29·0	...	26·0	55	"
218	"	29·2	...	26·0	50	"
219	"	29·4	...	25·0	50	"
220	"	28·9	...	28·0	45	"

[1] The details of these rollers are given in Figs. 405, 406, and a description will be found on p. 412.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
221	Roller castings	27·3	...	19·0	60	Sound
222	,,	31·3	...	19·0	50	,,
223	,,	33·5	11·0	...	45	Broke
224	,,	28·9	...	28·0	45	Sound
225	,,	29·1	12·0	...	50	,,
226	,,	27·3	...	19·0	60	,,
	Castings for machinery					
227	Cast steel [1]	32·9	14·0	...	50	Broke
228	Cylinders for hydraulic rams	31·8	13·0	...	90	Sound
229	,,	34·2	10·0	...	90	,,
230	,,	27·8	20·0 [2]	...	90	,,
231	,,	28·9	21·0	...	90	,,
232	,,	28·2	22·0 [2]	...	90	,,
233	,,	26·6	...	15·0	90	,,
234	,,	29·6	...	34·0	164	,,
235	,,	28·4	...	26·0	155	,,
236	,,	35·0	...	21·0	73	Broke
237	Pinions	33·0	...	25·0	85	,,
238	,,	34·7	...	27·0	90	Sound
239	,,	28·2	...	35·0	180	,,
240	,,	35·6	...	15·0	85	,,
241	,,	34·2	...	12·0	60	,,
242	Bevel pinions	31·6	...	8·5	55	...
243	,,	32·2	...	12·0 [3]	...	...
244	Wheels	36·4	...	22·0	83	,,
245	Clutches	30·2	...	12·0	78	Broke
246	,,	32·0	...	17·0	74	,,
247	,,	32·9	...	18·0	68	,,
248	Crank discs	28·2	...	35·0	164	,,
249	,,	29·6	...	34·0	73	,,
250	Stuffing-box covers	31·6	...	31·0	180	,,
251	,,	31·4	...	29·0	180	,,
252	,,	29·0	...	30·0	180	Sound

[1] These cylinders are referred to on p. 415.
[2] Extension on 6 inches.
[3] Re-test of No. 242.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
253	Sundry small castings	31·3	28·0	...	57	Broke
254	"	27·0	13·5	...	51	Sound
255	"	32·2	...	16·0	45	"
256	"	29·2	25·0	...	50	"
257	"	29·6	...	28·0	86	"
258	"	31·2	11·0	...	75	Broke
259	"	33·8	11·5	...	115	"
260	"	32·6	10·5	...	120	"
261	"	28·9	...	21·0	55	Sound
262	"	33·9	...	24·0	145	Broke
263	"	33·0	...	25·0	90	"
264	"	29·6	...	28·0	135	Sound
265	"	33·9	...	24·0	45	Broke
266	"	27·0	13·0	...	51	Sound
267	Brackets for sliding doors	30·5	18·5	...	...	...
268	"	36·0	15·7	...	...	...
269	"	29·8	17·5	...	...	...
270	Crucible cast-steel wheels for bogie waggons to carry 50-ton loads	35·0	...	30·0	...	On 2"
271		39·0	...	26·6	...	On 3"
272		36·5	...	29·3	...	"
273	Pinion	32·6	...	6·0 (test piece defective)	90	Sound
274	"	31·6	...	25·0	90	"
275	"	31·4	...	32·0	45	"
276	"	34·0	...	16·0	105	"
277	"	33·0	...	20·0	105	"
278	"	32·0	...	28·0	90	"
279	"	29·7	...	14·0	60	Broke
280	Bevel wheel	33·0	...	27·0	90	Sound
281	"	33·8	...	30·0	95	"
282	"	39·0	...	20·0	90	"
283	Running wheel	34·8	...	27·0	45	"
284	Curb ring	35·6	...	23·0	45	"
285	"	35·0	...	23·0	90	"
286	"	37·0	...	22·0	60	"
287	"	32·0	...	32·0	90	"
288	Sundry castings for various purposes	27·6	8·5	...	65	Broke

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
289	Sundry castings for various purposes	30·5	...	18·0	77	Broke
290	"	31·2	...	25·0	60	"
291	"	32·2	...	26·0	87	"
292	"	32·2	...	26·0	95	"
293	"	29·4	15·0	...	87	"
294	"	29·8	...	21·0	85	Sound
295	"	28·0	10·0	...	90	"
296	"	30·5	...	18·0	65	Broke
297	"	30·8	...	24·0	87	"
298	"	32·5	19·0	...	75	"
299	"	29·4	16·0	...	90	"
300	"	29·8	15·0	...	65	"
301	"	29·2	...	18·0	118	Sound
302	"	29·2	...	15·0	55	Broke
303	"	31·5	...	21·0	60	"
304	"	30·9	...	22·0	63	"
305	"	31·2	...	20·0	58	"
306	"	31·9	...	14·0	59	"

The following experiments on the transverse resistance to bending of cast-steel bars are of interest, more especially as similar tests are, so far as the writer knows, somewhat rare.

TABLE No. 21.

The Transverse Strength of Cast-steel Bars.

The bars were of the nominal dimensions of 2″ × 1″, planed all over, the load being applied at the centre of the specimen, and the greater dimension of the bar being vertical. Distance between supports = 36 inches.

No. of specimen.	Dimensions.	Span.	Load applied.		Deflection in inches.	Remarks.
			Pounds.	Tons.		
I.	1·958″ × 0·963″	3′ 0″	1120	0·5	0·06	Permanent set at 1·5 ton of 0·07 inch.
			2240	1·0	0·11	
			3360	1·5	0·21	

No. of specimen.	Dimensions.	Span.	Load applied.		Deflection in inches.	Remarks.
			Pounds.	Tons.		
II.	1·954″ × 0·978″	3′ 0″	1120 2240 3360	0·5 1·0 1·5	0·07 0·13 0·25	Permanent set at 1·5 ton of 0·07 inch.
III.	2·00″ × 1·00″	3′ 0″	1120 2240 3360 4480 5891	0·5 1·0 1·5 2·0 2·63	0·07 0·15 0·61 2·30	Permanent set at 1·0 ton of 0·03 inch. Specimen bent through 70°, but not broken; slight cracks visible.
IV.	2·05″ × 1·05″	3′ 0″	1120 2240 3360 4480 5980	0·5 1·0 1·5 2·0 2·67	0·09 0·15 0·25 1·59 Break	Permanent set at 1·0 ton of 0·02 inch. Specimen broke at 2·67 tons.
V.	2·00″ × 1·00″	3′ 0″	1120 2240 3360 4480 5600 6720	0·5 1·0 1·5 2·0 2·5 3·0	0·06 0·12 0·17 0·62 1·79 3·31	Permanent set not noted. Specimen bent through 64°, but did not break.

The chemical analyses of the bars referred to in the foregoing table are as follows :—

BARS NOS. I. AND II.

	Per cent.
Carbon	0·250
Silicon	0·086
Sulphur	0·073
Phosphorus	0·065
Manganese	0·911
Iron (by difference)	98·615
	100·000

Ultimate tensile strength = 31·2 tons per square inch.
Ultimate elongation = 10·0 per cent. in 8 inches.
Angle bent through (1 inch square) = 90°.

BARS NOS. III. AND IV.

	Per cent.
Carbon	0·265
Silicon	0·320
Sulphur	0·082
Phosphorus	0·042
Manganese	0·637
Iron (by difference)	98·654
	100·000

Ultimate tensile strength = 28·86 tons per square inch.
Ultimate elongation = 22·5 per cent. in 2 inches.
Contraction of area = 22·0 per cent.

BAR NO. V.

	Per cent.
Carbon	0·415
Silicon	0·187
Sulphur	0·067
Phosphorus	0·036
Manganese	0·511
Iron (by difference)	98·784
	100·000

Ultimate tensile strength = 35·6 tons per square inch.
Ultimate elongation = 15·6 per cent. in 2 inches.
Angle bent through (1 inch square) = 85°.

The chemical analyses of two specimens of cast-steel bars not submitted to a transverse test are as follows:—

	(a) Per cent.		(b) Per cent.
Carbon	0·232	... ...	0·237
Silicon	0·392	... ...	0·187
Sulphur	0·045	... ...	0·064
Phosphorus	0·045	... ...	0·042
Manganese	0·504	... ...	0·511
Iron (by difference) ...	98·782	... ...	98·959
	100·000		100·000

Ultimate tensile strength (*a*) = 28·9, (*b*) = 28·2 tons per square inch.

Ultimate elongation (*a*) = 21 per cent. in 8 inches, (*b*) = 35 per cent. in 2 inches.

Angle bent through (1 inch square) (*a*) = 90°, (*b*) = 180°.

Chemical analysis of cast-steel brackets for sliding doors—

	Per cent.
Carbon	0·488
Silicon	0·269
Sulphur	0·085
Phosphorus	0·070
Manganese	0·421
Iron (by difference)	98·667
	100·000

The ultimate tensile strength of this metal was 36·0 tons per square inch, and the ultimate elongation was 15·7 per cent. in 8 inches. (Test No. 268, Table No. 20.)

The series of tests which follows on the transverse strength of cast-iron bars is but a selection from a number of tests made in different foundries, and covering a wide range of constructive practice. The material represented was used for bridge and jetty cylinders, bollards, machinery castings, various details in connection with bridge construction, penstock frames and doors, and the like.

The variations shown are characteristic, but the general average indicates that the standard of strength and elasticity aimed at in the specification has been generally attained, and fairly represents good modern foundry practice in this class of work.

TABLE No. 22.

THE TRANSVERSE STRENGTH OF CAST-IRON TEST-BARS LOADED AT THE CENTRE, ON SUPPORTS 36 INCHES APART.

Nominal size of bar, 2 inches deep × 1 inch wide.

No. of Test.	Actual dimensions of test-bar as cast. Inches.		Actual breaking load at centre. Pounds.	Deflection at fracture. Inches.	Equivalent breaking load of 2" × 1" bar calculated in the proportion of BD^2. Pounds.
	Depth.	Breadth.			
1	2·06	1·06	3046	0·33	2715
2	2·08	1·07	2934	0·30	2540 [1]
3	2·11	1·05	3203	0·30	2744 [1]
4	2·06	1·07	4278	0·35	3770

[1] Flaw on underside.

No. of Test.	Actual dimensions of test-bar as cast. Inches. Depth.	Breadth.	Actual breaking load at centre. Pounds.	Deflection at fracture. Inches.	Equivalent breaking load of 2" × 1" bar calculated in the proportion of BD². Pounds.
5	2·06	1·06	3875	0·30	3452
6	2·1	1·1	2329	0·25	1921 [1]
7	2·06	1·07	5734	0·55	5040 [2]
8	2·04	1·02	4726	0·40	4457
9	2·07	1·09	4300	0·50	3683
10	2·07	1·10	3472	0·28	2947 [1]
11	2·06	1·08	3516	0·30	3069 [1]
12	2·08	1·06	5465	0·50	4768
13	2·07	1·07	2822	0·15	2462 [1]
14	2·06	1·08	4009	0·35	3500
15	2·07	1·06	4054	0·33	3500
16	2·05	1·06	3584	0·30	3218
17	2·10	1·07	4278	0·35	3626
18	2·00	1·00	3940	0·41	3940
19	2·00	1·00	3800	0·44	3800
20	2·00	1·00	3780	0·41	3780
21	2·00	1·00	3830	0·38	3830
22	2·00	1·00	4490	0·44	4490
23	2·00	1·00	4320	0·39	4320
24	2·01	1·06	3113	0·45	2909
25	2·02	1·00	3875	0·40	3800
26	2·00	1·00	3300	0·41	3300
27	2·00	1·00	3390	0·41	3390
28	2·00	1·00	3290	0·41	3290
29	2·00	1·00	3150	0·38	3150
30	2·00	1·00	3140	0·37	3140
31	2·00	1·00	3160	0·38	3160
32	2·00	1·00	4010	0·47	4010
33	2·00	1·00	3820	0·42	3820
34	2·00	1·00	3390	0·43	3390
35	2·00	1·00	3550	0·46	3550
36	2·00	1·00	3360	0·45	3360
37	2·00	1·00	3820	0·42	3820
38	2·00	1·00	3640	0·41	3640
39	2·00	1·00	3710	0·42	3710
40	2·00	1·03	4050	0·43	3927
41	2·00	1·00	4210	0·49	4210
42	2·00	1·03	3490	0·40	3384
43	2·00	1·00	3610	0·43	3610
44	2·00	1·00	3780	0·41	3780
45	2·00	1·00	3710	0·45	3710
46	2·00	1·00	3674	0·43	3674
47	2·00	1·00	3150	0·35	3150
48	2·00	1·00	3000	0·29	3000

[1] Flaw on underside. [2] Flaw on upper half.

No. of Test.	Actual dimensions of test-bar as cast. Inches.		Actual breaking load at centre. Pounds.	Deflection at fracture. Inches.	Equivalent breaking load of 2" × 1" bar calculated in the proportion of BD². Pounds.
	Depth.	Breadth.			
49	2·00	1·00	3350	0·40	3350
50	2·00	1·00	3100	0·30	3100
51	2·12	1·07	4233	0·35	3519
52	2·06	1·09	3785	0·30	3274
53	2·11	1·05	3248	0·25	2781 [1]
54	2·07	1·07	4278	0·35	3736
55	2·06	1·07	4188	0·32	3690
56	2·12	1·08	4435	0·40	3654
57	2·00	1·00	3303	0·28	3303
58	2·00	1·00	3472	0·30	3472
59	2·00	1·00	3610	0·33	3610
60	2·00	1·00	3582	0·36	3582
61	2·00	1·00	3580	0·415	3580
62	2·00	1·00	3990	0·454	3990
63	2·00	1·00	3200	0·390	3200
64	2·00	1·00	3530	0·416	3530
65	2·00	1·00	3889	0·455	3889
66	2·00	1·00	3950	0·450	3950
67	2·00	1·00	3580	0·437	3580
68	2·00	1·00	3750	0·455	3750
69	2·00	1·00	3640	0·437	3640
70	2·00	1·00	3528	0·375	3528
71	2·00	1·00	3696	0·437	3371
72	2·00	1·00	4032	0·437	3371
73	2·00	1·00	4032	0·437	3371
74	2·00	1·00	3360	0·437	3360
75	2·00	1·00	3528	0·375	3528
76	2·00	1·00	3808	0·437	3808
77	2·00	1·00	3584	0·375	3584
78	2·00	1·00	3528	0·375	3528
79	2·00	1·00	3360	0·437	3360
80	2·00	1·00	3416	0·375	3416
81	2·00	1·00	3584	0·437	3584
82	2·00	1·00	3304	0·375	3304
83	2·00	1·00	3584	0·437	3584
84	2·00	1·00	3360	0·375	3360
85	2·00	1·00	3426	0·437	3426
86	2·00	1·00	3696	0·375	3696
87	2·00	1·00	3864	0·375	3438
88	2·00	1·00	3528	0·375	3528
89	2·00	1·00	3584	0·375	3584
90	2·00	1·00	3696	0·500	3472
91	2·00	1·00	3192	0·343	2902
92	2·00	1·00	3472	0·406	3270 [2]

[1] Flaw on underside. [2] Retest of No. 91.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
221	Roller castings	27·3	...	19·0	60	Sound
222	"	31·3	...	19·0	50	"
223	"	33·5	11·0	...	45	Broke
224	"	28·9	...	28·0	45	Sound
225	"	29·1	12·0	...	50	"
226	"	27·3	...	19·0	60	"
	Castings for machinery					
227	Cast steel [1]	32·9	14·0	...	50	Broke
228	Cylinders for hydraulic rams	31·8	13·0	...	90	Sound
229	"	34·2	10·0	...	90	"
230	"	27·8	20·0 [2]	...	90	"
231	"	28·9	21·0	...	90	"
232	"	28·2	22·0 [2]	...	90	"
233	"	26·6	...	15·0	90	"
234	"	29·6	...	34·0	164	"
235	"	28·4	...	26·0	155	"
236	"	35·0	...	21·0	73	Broke
237	Pinions	33·0	...	25·0	85	"
238	"	34·7	...	27·0	90	Sound
239	"	28·2	...	35·0	180	"
240	"	35·6	...	15·0	85	"
241	"	34·2	...	12·0	60	"
242	Bevel pinions	31·6	...	8·5	55	...
243	"	32·2	...	12·0 [3]	...	...
244	Wheels	36·4	...	22·0	83	"
245	Clutches	30·2	...	12·0	78	Broke
246	"	32·0	...	17·0	74	"
247	"	32·9	...	18·0	68	"
248	Crank discs	28·2	...	35·0	164	"
249	"	29·6	...	34·0	73	"
250	Stuffing-box covers	31·6	...	31·0	180	"
251	"	31·4	...	29·0	180	"
252	"	29·0	...	30·0	180	Sound

[1] These cylinders are referred to on p. 415.
[2] Extension on 6 inches.
[3] Re-test of No. 242.

No. of test.	Description of casting.	Ultimate tensile strength. Tons per sq. inch.	Elongation in 8 inches. Per cent.	Elongation in 2 inches. Per cent.	Angle through which test-piece was bent. Degrees.	Remarks.
253	Sundry small castings	31·3	28·0	...	57	Broke
254	„	27·0	13·5	...	51	Sound
255	„	32·2	...	16·0	45	„
256	„	29·2	25·0	...	50	„
257	„	29·6	...	28·0	86	„
258	„	31·2	11·0	...	75	Broke
259	„	33·8	11·5	...	115	„
260	„	32·6	10·5	...	120	„
261	„	28·9	...	21·0	55	Sound
262	„	33·9	...	24·0	145	Broke
263	„	33·0	...	25·0	90	„
264	„	29·6	...	28·0	135	Sound
265	„	33·9	...	24·0	45	Broke
266	„	27·0	13·0	...	51	Sound
267	Brackets for sliding doors	30·5	18·5	...	...	...
268	„	36·0	15·7	...	...	...
269	„	29·8	17·5	...	...	...
270	Crucible cast-steel wheels for bogie waggons to carry 50-ton loads	35·0	...	30·0	...	On 2″
271		39·0	...	26·6	...	On 3″
272		36·5	...	29·3	...	„
273	Pinion	32·6	...	6·0 (test piece defective)	90	Sound
274	„	31·6	...	25·0	90	„
275	„	31·4	...	32·0	45	„
276	„	34·0	...	16·0	105	„
277	„	33·0	...	20·0	105	„
278	„	32·0	...	28·0	90	„
279	„	29·7	...	14·0	60	Broke
280	Bevel wheel	33·0	...	27·0	90	Sound
281	„	33·8	...	30·0	95	„
282	„	39·0	...	20·0	90	„
283	Running wheel	34·8	...	27·0	45	„
284	Curb ring	35·6	...	23·0	45	„
285	„	35·0	...	23·0	90	„
286	„	37·0	...	22·0	60	„
287	„	32·0	...	32·0	90	„
288	Sundry castings for various purposes	27·6	8·5	...	65	Broke

CHAPTER II.

ROLLED SECTIONS IN STEEL AND THEIR MECHANICAL ELEMENTS, WITH GENERAL REMARKS ON THEIR USES AND COMBINATIONS.

Angles—Equal-legged—Unequal-legged—Round-backed—Acute-angled—Obtuse-angled—Bulb-angles—British standard sections—Table of the principal mechanical elements of equal-legged angles; of unequal-legged angles—Tees—British standard sections—Bulb tees—Table of the principal mechanical elements of ordinary tees; of bulb tees—Rolled joists—General remarks—Proportions of web and flange thicknesses—Standards of proportion—British standard sections—Table of the principal mechanical elements of rolled joists—Channels—Standards of proportions—British standard sections—Table of the principal mechanical elements of channels—Zed angles—General remarks—British standard sections—Table of the principal mechanical elements of Zed angles—Other forms of sections—Plates—Bars—Flats.

It is proposed in this chapter to give a short description of the principal rolled sections in steel commonly used in ordinary

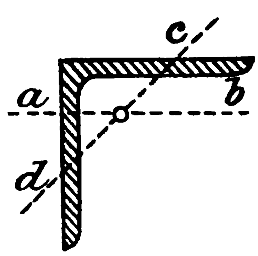


Fig. 7.

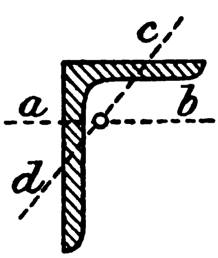


Fig. 8.

riveted construction, with tables of their principal mechanical elements, and some practical remarks on their use.

Angles.—The angle-steel, or, to use the older nomenclature, the angle-iron, is perhaps the most commonly in use among all those sections of material which go to make up riveted work. It may be "equal-legged," as in Fig. 7, or "unequal-legged," as in Fig. 8.

The equal-legged angle in its ordinary form has a rectangular outline with a square corner on the outside, the interior faces being sometimes slightly tapered with a connecting round in the inner corner, and the edges rounded off to a quadrant of small radius.

These tapers and the radii of the roundings are not quite the same in all section books, varying with the shape of the rolls of the respective makers, although proposals for the adoption of a uniform standard in this as in other sections have not been wanting.

These proposals have now assumed a definite form in this country, by the issue of the "British Standard Sections," compiled under the direction of the Engineering Standards Committee in 1904, and to these sections the attention of the student is directed; and in the work entitled "Properties of British Standard Sections" will be found the standard sizes, thicknesses, slopes of taper, and radii of connecting curves, together with tables of the mechanical elements of the standard sections.

The unequal-legged angle presents the same general characteristics, while its name speaks for itself. Both the equal- and unequal-legged section is, in the British standard section, of uniform thickness, without taper.

Variations from these forms are found in the acute-angled angle and the obtuse-angled angle, used where oblique connections of riveted work have to be made. The use of acute-angled angles is attended sometimes with the difficulty of getting the rivets into the acute angle, which must be borne in mind in these cases, it being sometimes necessary, if the angle of connection is very acute, to use a bent plate of sufficient dimensions in lieu of an angle.

Both equal- and unequal-legged angles are also rolled with a round back, as in Fig. 9. They are most commonly used to effect the splice in the main angles of plate or lattice girders, an example of which is given in Fig. 101, the round back of the connecting angle fitting into the interior rounding of the main angles to be spliced. This method of splicing the main angles of a riveted plate girder is the one most commonly adopted, and leads to the consideration of the net sectional area of the angles to be spliced, the corresponding thickness of the angle "cover," which

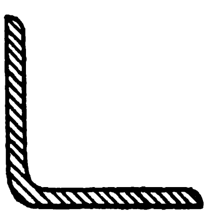


FIG. 9.

must necessarily be greater than that of the main angles, and as a consequence the spaces left for the rivets, their heads, and the amount of metal left outside them.

When, however, angles are used for ties or struts, either singly or in pairs, as in roof trusses, it becomes a simple matter to splice each leg of the angle with a flat of suitable width and thickness.

Another variation in the equal- or unequal-legged angle is that in which the legs are rolled of equal thickness without taper, and the edges and corners, both internal and external, are square and sharp. This, however, is usually considered a special section, and not frequently adopted in ordinary riveted work.

A section of angle-iron used frequently in the frames of ship or caisson work, as beams subject to transverse stress, is the so-called bulb-angle, shown in Fig. 10. This angle is usually unequal-legged, the object of the bulb being to increase the moment of inertia of the beam in the plane of its greatest depth as a beam, while it also serves the purpose of thickening and rounding the edge of the angle where exposed to passing traffic, etc.

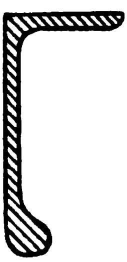


Fig. 10.

The bulb may be rolled on one side of the longest limb as shown, or on the opposite side. The sectional area of the bulb varies slightly with different makers, but is standardized in the British standard section.

According to the usual practice, the vertical limb or web with the bulb is made parallel, the taper being given to the other limb of the angle. In the British standard section of bulb-angle, both limbs are of uniform thickness, there being no taper. If the section be increased beyond a certain minimum thickness the dimensions of the sides are increased proportionately, while the bulb retains the same projection from the face of the web.

The uses of angle-steels are multifarious. In addition to their primary function of connecting members of a structure in planes at right angles to one another, such as the web and flanges of a plate girder, or in oblique connections, they are also found used to a great extent either as beams, struts, or ties. As beams we find them in purlins to roofs, as secondary beams in a variety of structures, in the framing to sides of corrugated iron sheds, and the like. As struts they are employed in the members of

lattice girders, in the compression members of roof principals, and as secondary bracing where some lateral stiffness is required in a number of cases. As ties they appear more or less successfully in the tension members of light trusses, but their effective use in tension is somewhat qualified by the necessity of securing them in many cases by one leg only. This remark also applies under similar conditions to their use as struts. In this latter capacity they will be further referred to in the chapter on columns.

The selection of the dimensions and scantlings of angles will be determined by a variety of considerations depending on the use to which they are put. As connecting members simply we shall find their dimensions ruled to a large extent by the size of rivets employed, the bearing stress allowed, and so on; as beams, by their moment of inertia; as struts, by their least radius of gyration. But in most cases considerations of rivet spacing in connections, etc., will be ruling factors in the design; and the young draughtsman will in this, as in so many other cases, be wise in drawing all doubtful details full size, or to a large scale, before he finally determines his section.

The following tables give the dimensions, weight per foot run, sectional area, moments of inertia, and least radii of gyration for equal-legged, unequal-legged, and bulb-angles in steel:—

TABLE No. 23.

THE PRINCIPAL MECHANICAL ELEMENTS OF EQUAL-LEGGED ANGLES [1] (See Fig. 7).

Section. Equal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis *a—b*, Fig. 7.	Approximate least radius of gyration, axis *c—d*.	Distance of axis *a–b* from farthest edge of section. Inches.
8″ × 8″ × 1″	15·00	51·00	88·98	1·56	5·63
8″ × 8″ × $\frac{7}{8}$″	13·23	44·98	79·46	...	5·68
8″ × 8″ × $\frac{3}{4}$″	11·44	38·89	70·04	1·57	5·72
8″ × 8″ × $\frac{5}{8}$″	9·61	32·67	59·80	...	5·77
8″ × 8″ × $\frac{1}{2}$″	7·75	26·35	48·65	1·58	5·87
7″ × 7″ × 1″	13·00	44·20	58·16	1·36	4·89

[1] In this and the following tables the effects of taper and of the circular curves connecting members are ignored. The values are therefore approximate, but of sufficient accuracy for practical purposes.

Section. Equal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis a—b, Fig. 7.	Approximate least radius of gyration, axis c—d.	Distance of axis a—b from farthest edge of section. Inches.
$7'' \times 7'' \times \frac{7}{8}''$	11·48	39·05	52·10	...	4·93
$7'' \times 7'' \times \frac{3}{4}''$	9·94	33·79	45·86	1·37	4·98
$7'' \times 7'' \times \frac{5}{8}''$	8·36	28·42	39·14	...	5·02
$7'' \times 7'' \times \frac{1}{2}''$	6·75	22·95	32·00	1·38	5·06
$6'' \times 6'' \times 1''$	11·00	37·40	35·46	1·18	4·13
$6'' \times 6'' \times \frac{7}{8}''$	9·73	33·10	31·88	...	4·18
$6'' \times 6'' \times \frac{3}{4}''$	8·44	28·69	28·12	1·19	4·22
$6'' \times 6'' \times \frac{5}{8}''$	7·11	24·17	24·11	...	4·27
$6'' \times 6'' \times \frac{1}{2}''$	5·75	19·55	19·90	1·20	4·31
$5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{7}{8}''$	8·86	30·12	24·07	1·06	3·81
$5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{3}{4}''$	7·69	26·14	21·32	...	3·85
$5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{5}{8}''$	6·48	22·05	18·03	...	3·90
$5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{1}{2}''$	5·25	17·85	15·19	1·08	3·94
$5'' \times 5'' \times \frac{3}{4}''$	6·94	23·59	15·75	0·98	3·48
$5'' \times 5'' \times \frac{5}{8}''$	5·86	19·92	13·56	...	3·52
$5'' \times 5'' \times \frac{1}{2}''$	4·75	16·15	11·24	1·00	3·56
$4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$	5·23	17·80	9·72	0·87	3·14
$4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$	4·25	14·45	8·00	...	3·19
$4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{7}{16}''$	3·75	12·74	7·18	...	3·21
$4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$	3·23	11·00	6·27	0·88	3·23
$4'' \times 4'' \times \frac{5}{8}''$	4·61	15·67	6·65	0·78	2·77
$4'' \times 4'' \times \frac{1}{2}''$	3·75	12·75	5·56	...	2·81
$4'' \times 4'' \times \frac{7}{16}''$	3·31	11·25	4·96	...	2·84
$4'' \times 4'' \times \frac{3}{8}''$	2·86	9·72	4·35	0·79	2·86
$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	3·98	13·55	4·32	0·68	2·40
$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	3·25	11·05	3·64	...	2·44
$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	2·87	9·76	3·24	...	2·46
$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	2·48	8·45	2·88	0·69	2·48
$3\frac{1}{4}'' \times 3\frac{1}{4}'' \times \frac{1}{2}''$	3·00	10·20	2·85	0·64	2·25
$3\frac{1}{4}'' \times 3\frac{1}{4}'' \times \frac{7}{16}''$	2·65	9·02	2·58	...	2·28
$3\frac{1}{4}'' \times 3\frac{1}{4}'' \times \frac{3}{8}''$	2·30	7·82	2·25	0·65	2·30
$3'' \times 3'' \times \frac{1}{2}''$	2·75	9·35	2·22	0·58	2·07
$3'' \times 3'' \times \frac{7}{16}''$	2·43	8·27	1·99	...	2·09
$3'' \times 3'' \times \frac{3}{8}''$	2·11	7·17	1·75	0·59	2·11
$2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{1}{2}''$	2·50	8·50	1·67	0·53	1·87
$2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{7}{16}''$	2·21	7·53	1·51	...	1·90
$2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$	1·92	6·53	1·33	0·54	1·92
$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$	2·00	6·79	1·11	0·49	1·71
$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	1·73	5·90	0·98	...	1·74
$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$	1·47	4·98	0·85	0·50	1·76
$2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{3}{8}''$	1·55	5·26	0·70	0·44	1·55
$2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{5}{16}''$	1·31	4·45	0·61	0·45	1·57
$2'' \times 2'' \times \frac{3}{8}''$	1·36	4·62	0·48	0·39	1·36
$2'' \times 2'' \times \frac{5}{16}''$	1·15	3·92	0·42	0·40	1·38
$1\frac{3}{4}'' \times 1\frac{3}{4}'' \times \frac{5}{16}''$	1·00	3·39	0·27	0·34	1·20

Section. Equal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis *a*—*b*, Fig. 7.	Approximate least radius of gyration, axis *c*—*d*.	Distance of axis *a*—*b* from farthest edge of section. Inches.
1¾" × 1¾" × ¼"	0·81	2·76	0·23	0·34	1·22
1½" × 1½" × 5/16"	0·84	2·86	0·16	0·30	1·01
1½" × 1½" × ¼"	0·69	2·34	0·14	0·30	1·03
1¼" × 1¼" × ¼"	0·56	1·91	0·077	0·24	0·85
1¼" × 1¼" × ⅛"	0·29	0·98	0·044	0·24	0·89
1" × 1" × ¼"	0·44	1·49	0·037	0·19	0·66
1" × 1" × ⅛"	0·23	0·78	0·022	0·19	0·70
¾" × ¾" × ⅛"	0·17	0·58	0·008	0·15	0·51

TABLE No. 24.

The Principal Mechanical Elements of Unequal-legged Angles (See Fig. 8).

Section. Unequal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis *a*—*b*, Fig. 8.	Approximate least radius of gyration about axis *c*—*d*.	Distance of axis *a*—*b* from farthest edge of section. Inches.
14" × 3½" × ½"	8·50	28·90	160·0	0·94	8·19
11' × 3" × ½"	6·75	22·95	78·5	0·81	6·47
10" × 3½" × ½"	6·50	22·10	63·5	0·92	6·10
9" × 3½" × ½"	6·00	20·40	48·0	0·90	5·56
8" × 4½" × ¾"	8·81	29·96	57·1	0·96	5·16
8" × 4½" × ⅝"	7·42	25·23	48·6	0·96	5·21
8" × 4½" × ½"	6·00	20·40	39·8	0·96	5·25
8" × 3½" × ⅝"	6·80	23·11	42·5	0·73	4·96
8" × 3½" × ½"	5·50	18·70	33·5	0·73	5·02
8" × 3" × ⅝"	6·48	22·05	40·2	0·73	4·84
8" × 3" × ½"	5·25	17·85	32·6	0·73	4·89
7½" × 3" × ⅝"	6·17	20·98	33·7	0·71	4·57
7½" × 3" × ½"	5·00	17·00	27·5	0·71	4·62
7" × 4" × ¾"	7·69	26·14	37·87	0·85	4·50
7" × 4" × ⅝"	6·48	22·05	32·24	0·85	4·54
7" × 4" × ½"	5·25	17·85	26·62	0·86	4·58
7" × 3½" × ⅝"	6·17	20·98	30·73	0·75	4·43
7" × 3½" × ½"	5·00	17·00	25·33	0·75	4·47
7" × 3" × ⅝"	5·86	19·92	27·9	0·76	4·30
7" × 3" × ½"	4·75	16·15	22·9	0·76	4·35
7" × 3" × ⅜"	3·61	12·27	17·6	0·77	4·40
6½" × 4½" × ⅝"	6·41	22·05	27·56	0·95	4·34
6½" × 4½" × ½"	5·25	17·85	23·49	0·96	4·38
6½" × 4½" × ⅜"	3·98	13·55	17·68	0·96	4·43
6½" × 4" × ⅝"	6·17	20·98	26·36	0·84	4·26

Section. Unequal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis a—b, Fig. 8.	Approximate least radius of gyration about axis c—d.	Distance of axis $a-b$ from farthest edge of section. Inches.
$6\frac{1}{2}'' \times 4'' \times \frac{1}{2}''$	5·00	17·00	21·42	0·85	4·30
$6\frac{1}{2}'' \times 4'' \times \frac{3}{8}''$	3·80	12·91	16·81	0·85	4·34
$6\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	5·86	19·92	24·99	0·79	4·15
$6\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	4·75	16·15	20·37	0·80	4·19
$6\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	3·61	12·27	16·15	0·80	4·24
$6\frac{1}{2}'' \times 3'' \times \frac{5}{8}''$	5·55	18·86	24·35	0·78	4·03
$6\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	4·50	15·30	18·75	0·79	4·08
$6\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	3·42	11·63	14·35	0·79	4·13
$6'' \times 5'' \times \frac{7}{8}''$	8·86	30·12	30·13	1·03	4·04
$6'' \times 5'' \times \frac{3}{4}''$	7·69	26·14	26·46	1·04	4·08
$6'' \times 5'' \times \frac{5}{8}''$	6·48	22·05	22·68	1·04	4·13
$6'' \times 4'' \times \frac{3}{4}''$	6·94	23·59	24·62	0·87	3·92
$6'' \times 4'' \times \frac{5}{8}''$	5·86	19·92	21·25	0·88	3·96
$6'' \times 4'' \times \frac{1}{2}''$	4·75	16·15	17·45	0·88	4·01
$6'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	5·55	18·86	20·18	0·79	3·87
$6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	4·50	15·30	16·63	0·80	3·92
$6'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	3·97	13·48	14·74	0·80	3·94
$6'' \times 3'' \times \frac{5}{8}''$	5·23	17·80	18·92	0·76	3·76
$6'' \times 3'' \times \frac{1}{2}''$	4·25	14·45	15·68	0·77	3·81
$6'' \times 3'' \times \frac{3}{8}''$	3·23	11·00	12·12	0·77	3·85
$5\frac{1}{2}'' \times 4'' \times \frac{5}{8}''$	5·55	18·86	16·37	0·86	3·67
$5\frac{1}{2}'' \times 4'' \times \frac{1}{2}''$	4·50	15·30	13·64	0·87	3·72
$5\frac{1}{2}'' \times 4'' \times \frac{3}{8}''$	3·42	11·63	10·51	0·87	3·78
$5\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	5·23	17·80	15·84	0·79	3·59
$5\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	4·25	14·45	13·04	0·80	3·63
$5\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	3·23	11·00	10·07	0·80	3·67
$5\frac{1}{2}'' \times 3'' \times \frac{5}{8}''$	4·92	16·73	14·91	0·67	3·49
$5\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	4·00	13·60	12·28	0·68	3·53
$5\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	3·05	10·36	9·53	0·68	3·57
$5'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$	5·55	18·86	13·10	0·89	3·45
$5'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$	4·50	15·30	10·88	0·90	3·50
$5'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$	3·42	11·63	8·44	0·90	3·54
$5'' \times 4'' \times \frac{5}{8}''$	5·23	17·80	12·55	0·85	3·38
$5'' \times 4'' \times \frac{1}{2}''$	4·25	14·45	10·45	0·86	3·42
$5'' \times 4'' \times \frac{3}{8}''$	3·23	11·00	8·12	0·86	3·46
$5'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	4·92	16·73	12·03	0·75	3·30
$5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	4·00	13·60	10·02	0·76	3·34
$5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	3·05	10·36	7·79	0·76	3·39
$5'' \times 3'' \times \frac{5}{8}''$	4·61	15·67	11·36	0·65	3·21
$5'' \times 3'' \times \frac{1}{2}''$	3·75	12·75	9·45	0·66	3·25
$5'' \times 3'' \times \frac{3}{8}''$	2·86	9·72	7·31	0·66	3·29
$4\frac{1}{2}'' \times 4'' \times \frac{5}{8}''$	4·92	16·73	9·37	0·80	3·08
$4\frac{1}{2}'' \times 4'' \times \frac{1}{2}''$	4·00	13·60	7·87	0·81	3·13
$4\frac{1}{2}'' \times 4'' \times \frac{3}{8}''$	3·05	10·36	6·05	0·81	3·17
$4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	4·61	15·67	8·83	0·74	3·01

Section. Unequal-legged angles.	Area in sq. inches.	Weight in lbs. per foot lineal.	Approximate moment of inertia about axis *a—b*, Fig. 8.	Approximate least radius of gyration about axis *c—d*.	Distance of axis *a—b* from farthest edge of section. Inches.
$4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	3·75	12·75	7·33	0·75	3·05
$4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	2·86	9·72	5·71	0·75	3·09
$4\frac{1}{2}'' \times 3'' \times \frac{5}{8}''$	4·30	14·61	8·50	0·65	2·91
$4\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	3·50	11·90	7·02	0·66	2·96
$4\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	2·67	9·08	5·49	0·66	3·00
$4\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	3·25	11·05	6·63	0·62	2·86
$4\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	2·48	8·45	5·17	0·62	2·90
$4'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	4·30	14·61	6·36	0·72	2·70
$4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	3·50	11·90	5·31	0·73	2·75
$4'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	2·67	9·08	4·16	0·73	2·79
$4'' \times 3'' \times \frac{5}{8}''$	3·98	13·55	6·01	0·64	2·63
$4'' \times 3'' \times \frac{1}{2}''$	3·25	11·05	5·03	0·65	2·67
$4'' \times 3'' \times \frac{3}{8}''$	2·48	8·45	3·95	0·65	2·71
$4'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	3·00	10·20	4·72	0·56	2·58
$4'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	2·30	7·81	3·79	0·56	2·62
$3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	3·00	10·20	3·45	0·63	2·37
$3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$	2·30	7·81	2·71	0·63	2·42
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	2·75	9·35	3·25	0·55	2·29
$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	2·11	7·17	2·57	0·55	2·33
$3\frac{1}{2}'' \times 2\frac{1}{4}'' \times \frac{3}{8}''$	2·02	6·85	2·46	0·53	2·29
$3\frac{1}{2}'' \times 2\frac{1}{4}'' \times \frac{5}{16}''$	1·70	5·78	2·10	0·53	2·32
$3'' \times 2\frac{3}{4}'' \times \frac{1}{2}''$	2·63	8·93	2·16	0·55	2·03
$3'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$	2·02	6·85	1·78	0·55	2·08
$3'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	2·50	8·50	2·09	0·53	2·00
$3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	1·92	6·53	1·64	0·53	2·04
$3'' \times 2'' \times \frac{5}{16}''$	1·47	4·98	1·32	0·44	1·98
$3'' \times 2'' \times \frac{1}{4}''$	1·19	4·04	1·07	0·44	2·01
$2\frac{1}{2}'' \times 2\frac{1}{4}'' \times \frac{5}{16}''$	1·39	4·72	0·82	0·45	1·73
$2\frac{1}{2}'' \times 2\frac{1}{4}'' \times \frac{1}{4}''$	1·13	3·83	0·68	0·45	1·75
$2\frac{1}{2}'' \times 2'' \times \frac{5}{16}''$	1·31	4·45	0·79	0·43	1·69
$2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$	1·06	3·61	0·65	0·43	1·71
$2\frac{1}{4}'' \times 1\frac{3}{4}'' \times \frac{1}{4}''$	0·94	3·19	0·46	0·34	1·52
$2'' \times 1\frac{3}{4}'' \times \frac{1}{4}''$	0·88	2·98	0·33	0·35	1·37
$2'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$	0·81	2·76	0·31	0·32	1·34
$2'' \times 1'' \times \frac{3}{16}''$	0·53	1·79	0·21	0·25	1·26
$1\frac{3}{4}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$	0·75	2·55	0·21	0·29	1·19
$1\frac{3}{4}'' \times 1\frac{1}{2}'' \times \frac{1}{8}''$	0·39	1·32	0·12	0·29	1·23
$1\frac{1}{2}'' \times 1\frac{1}{4}'' \times \frac{1}{4}''$	0·63	2·13	0·13	0·26	1·00
$1\frac{1}{2}'' \times 1\frac{1}{4}'' \times \frac{1}{8}''$	0·33	1·12	0·070	0·26	1·04
$1\frac{1}{4}'' \times 1'' \times \frac{1}{8}''$	0·26	0·88	0·040	0·21	0·85
$1\frac{1}{4}'' \times \frac{7}{8}'' \times \frac{1}{8}''$	0·25	0·85	0·039	0·20	0·83
$1\frac{1}{8}'' \times \frac{7}{8}'' \times \frac{1}{8}''$	0·23	0·78	0·028	0·20	0·76
$1'' \times \frac{3}{4}'' \times \frac{1}{8}''$	0·20	0·68	0·019	0·16	0·67
$\frac{3}{4}'' \times \frac{1}{2}'' \times \frac{1}{8}''$	0·14	0·47	0·007	0·11	0·48

Tees.—The tee-steel, or tee-iron, ranks perhaps next to the angle in general utility. Its general form is shown in Fig. 11.

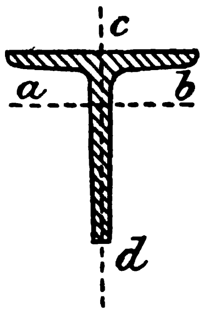


FIG. 11.

The proportions of top table to stem or web are very variable, and the error of misdescription of the dimensions is one very frequently found on drawings, rectified, it may be, by dimensioning the members, but the young draughtsman will do well to remember that a 6″ × 3″ tee is by no means the same thing as a 3″ × 6″ tee. He will probably ascertain this to his cost if he specifies the one in mistake for the other, in the absence of a figured section. The width of top table is the dimension first quoted.

The top table and stem are usually both slightly tapered, and connected by roundings of small radius, the corners of the extremities of the limbs being sometimes rounded and sometimes square. A variation sometimes found is when the top table is of uniform thickness and the stem tapered, or *vice versâ.*

The British standard section of tee has the top table and the web tapered, the edges of the top table being rounded off beneath, while the edge of the web is square.

Tees are commonly used as beams, as in the case of purlins, secondary bearers in fire-proof floors, and the like. As struts they are a favourite section for the compression members of roof trusses of moderate span, lattice girders, etc., also as stiffeners to the webs of plate girders. As ties their use is more limited, there being some difficulty in making such an end connection as will effectively bring into play the whole cross-section of the metal. Tee struts will be further referred to in the chapter on columns.

The proportions of tees to be adopted in any particular detail will, apart from the value of their moments of inertia when used as beams, or of their least radius of gyration when used as struts, be frequently ruled by the dimensions and spacing of their riveted or bolted connections. Thus, to take a familiar example, the tee stiffener to the web of a plate girder will require a width of top table or flange sufficient to take the rivets required in a joint of the web plating, which again will be ruled by the shearing stresses in the web, and the number and diameter of the rivets required. Or supposing, in the case, let us say, of a footbridge with timber floor secured to tee bearers by bolts or coach-screws, the

top table of the tee must be of width enough to receive screws or bolts of the diameter required, with (*a*) a sufficient amount of metal outside the hole, and (*b*) sufficient space between the stem and hole for the nut of the bolt or head of the coach-screw. Such elementary considerations may bear the aspect of truisms, but careful attention to points of detail such as these will always be found to characterize sound ironwork design.

Bulb Tees.—A tee section with a bulb rolled on the lower extremity of the stem constitutes the useful section known as "bulb tee" or "deck beam." This section is used to a considerable extent in shipbuilding, and occasionally in purlins and similar beams. The moment of inertia is increased by the bulb, which also forms a finish to the lower edge of the stem, which is usually rolled with parallel sides, the top table or flange having a taper similar to the flange of a rolled joist. The area of the bulb and relative thickness of stem and flange vary somewhat in different rolling mills, but are standardized in the British standard section.

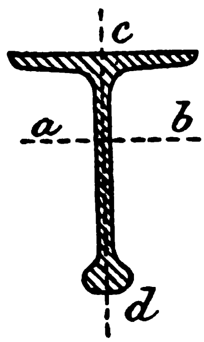


FIG. 12.

The following tables give the dimensions, sectional area, weight per foot run, greatest moment of inertia, position of centre of gravity, and least radius of gyration, for ordinary tees and for bulb tees of average proportions of stem, flange, and bulbs.

TABLE No. 25.

THE PRINCIPAL MECHANICAL ELEMENTS OF ORDINARY TEES (See Fig. 11).

Section. Ordinary tees. Table × web.	Area in sq. inches.	Weight in lbs. per foot run.	Greatest moment of inertia about axis *a*—*b*. Fig. 11.	Distance of centre of gravity above lower edge.	Approximate least radius of gyration about axis *c*—*d*.
7″ × 3½″ × ¾″	7·31	24·86	6·12	2·63	...
7″ × 3½″ × ⅝″	6·17	20·98	5·28	2·68	...
7″ × 3½″ × ½″	5·00	17·00	4·30	2·72	...
7″ × 3½″ × 7/16″	4·40	14·97	3·96	2·75	1·65
7″ × 3⅛″ × ⅝″	5·94	20·19	3·83	2·40	...
7″ × 3⅛″ × ½″	4·81	16·36	3·17	2·44	1·62
6½″ × 6½″ × ¾″	9·19	31·24	35·16	4·61	...

Section. Ordinary tees. Table × web.	Area in sq. inches.	Weight in lbs. per foot run.	Greatest moment of inertia about axis *a*—*b*. Fig. 11.	Distance of centre of gravity above lower edge.	Approximate least radius of gyration about axis *c*—*d*.
6½″ × 6½″ × ⅝″	7·73	26·30	31·06	4·64	...
6½″ × 6½″ × ½″	6·25	21·25	25·70	4·68	1·50
6½″ × 6″ × ¾″	8·81	29·96	28·78	4·28	...
6½″ × 6″ × ⅝″	7·42	25·23	24·71	4·33	...
6½″ × 6″ × ½″	6·00	20·40	20·36	4·38	1·50
6″ × 5″ × ¾″	7·69	26·14	16·72	3·59	...
6″ × 5″ × ⅝″	6·48	22·05	14·40	3·63	...
6″ × 5″ × ½″	5·25	17·85	11·92	3·68	...
6″ × 5″ × ⅜″	3·98	13·55	9·22	3·72	1·33
6″ × 4″ × ⅝″	5·86	19·92	7·53	2·97	...
6″ × 4″ × ½″	4·75	16·15	6·20	3·02	...
6″ × 4″ × ⅜″	3·61	12·27	4·91	3·06	1·34
6″ × 3½″ × ⅝″	5·55	18·86	5·09	2·72	...
6″ × 3½″ × ½″	4·50	15·30	4·27	2·67	1·38
6″ × 3½″ × ⅜″	3·42	11·63	3·34	2·71	1·38
6″ × 3″ × ⅝″	5·23	17·80	3·22	2·26	...
6″ × 3″ × ½″	4·25	14·45	2·70	2·31	...
6″ × 3″ × ⅜″	3·23	11·00	2·10	2·35	1·40
5″ × 4″ × ⅝″	5·23	17·80	7·14	2·88	...
5″ × 4″ × ½″	4·25	14·45	5·76	2·93	...
5″ × 4″ × ⅜″	3·23	11·00	4·64	2·97	1·10
5″ × 3½″ × ⅝″	4·92	16·73	4·82	2·55	...
5″ × 3½″ × ½″	4·00	13·60	4·00	2·59	...
5″ × 3½″ × ⅜″	3·05	10·36	3·19	2·64	1·12
5″ × 3″ × ⅝″	4·61	15·67	3·06	2·20	...
5″ × 3″ × ½″	3·75	12·75	2·59	2·25	...
5″ × 3″ × ⅜″	2·86	9·72	2·00	2·29	1·14
5″ × 2½″ × ½″	3·50	11·90	1·51	1·89	...
5″ × 2½″ × ⅜″	2·67	9·08	1·19	1·94	1·16
4½″ × 4½″ × ⅝″	5·23	17·80	9·72	3·14	...
4½″ × 4½″ × ½″	4·25	14·45	8·00	3·19	...
4½″ × 4½″ × ⅜″	3·23	11·00	6·27	3·23	0·88
4½″ × 3½″ × ½″	3·75	12·75	3·92	2·55	...
4½″ × 3½″ × 7/16″	3·31	11·25	3·51	2·57	...
4½″ × 3½″ × ⅜″	2·86	9·72	3·07	2·60	0·90
4½″ × 2½″ × ½″	3·25	11·05	1·47	1·83	...
4½″ × 2½″ × 7/16″	2·87	9·76	1·32	1·85	...
4½″ × 2½″ × ⅜″	2·48	8·45	1·17	1·88	0·92
4″ × 5″ × ½″	4·25	14·45	10·45	3·42	...
4″ × 5″ × ⅜″	3·23	11·00	8·00	3·46	0·76
4″ × 4½″ × ½″	4·00	13·60	7·80	3·12	...
4″ × 4½″ × ⅜″	3·05	10·36	6·08	3·17	0·80
4″ × 4″ × ⅝″	4·61	15·67	6·65	2·77	...
4″ × 4″ × ½″	3·75	12·75	5·56	2·81	...
4″ × 4″ × ⅜″	2·86	9·72	4·20	2·86	0·82

Section. Ordinary tees. Table × web.	Area in sq. inches.	Weight in lbs. per foot run.	Greatest moment of inertia about axis *a—b*. Fig. 11.	Distance of centre of gravity above lower edge.	Approximate least radius of gyration about axis *c—d*.
4″ × 3½″ × ½″	3·50	11·90	3·81	2·50	...
4″ × 3½″ × 7/16″	3·09	10·51	3·41	2·52	...
4″ × 3½″ × ⅜″	2·67	9·08	2·98	2·54	0·86
4″ × 3″ × ½″	3·25	11·05	2·43	2·17	...
4″ × 3″ × 7/16″	2·87	9·76	2·18	2·19	...
4″ × 3″ × ⅜″	2·48	8·45	1·90	2·22	0·88
4″ × 2½″ × ½″	3·00	10·20	1·42	1·83	...
4″ × 2½″ × 7/16″	2·65	9·02	1·28	1·85	...
4″ × 2½″ × ⅜″	2·30	7·81	1·13	1·88	0·90
3½″ × 3½″ × ½″	3·25	11·05	3·64	2·44	...
3½″ × 3½″ × 7/16″	2·87	9·76	3·24	2·46	...
3½″ × 3½″ × ⅜″	2·48	8·45	2·80	2·48	0·72
3½″ × 3″ × ½″	3·00	10·20	2·33	2·12	...
3½″ × 3″ × 7/16″	2·65	9·02	2·22	2·14	...
3½″ × 3″ × ⅜″	2·30	7·81	1·84	2·17	0·74
3½″ × 2½″ × 7/16″	2·43	8·27	1·23	1·82	...
3½″ × 2½″ × ⅜″	2·11	7·17	1·09	1·84	...
3½″ × 2½″ × 5/16″	1·78	6·04	0·93	1·86	0·76
3″ × 3½″ × 7/16″	2·65	9·02	3·08	2·40	...
3″ × 3½″ × ⅜″	2·30	7·81	2·71	2·42	0·60
3″ × 3″ × 7/16″	2·43	8·27	1·99	2·09	...
3″ × 3″ × ⅜″	2·11	7·17	1·75	2·11	...
3″ × 3″ × 5/16″	1·78	6·04	1·50	2·08	0·62
3″ × 2½″ × 7/16″	2·21	7·53	1·18	1·77	...
3″ × 2½″ × ⅜″	1·92	6·53	1·04	1·79	...
3″ × 2½″ × 5/16″	1·62	5·51	0·87	1·82	0·64
3″ × 1½″ × ⅜″	1·55	5·26	0·23	1·11	...
3″ × 1½″ × 5/16″	1·31	4·45	0·20	1·13	0·68
2½″ × 2½″ × ⅜″	1·73	5·90	0·98	1·74	...
2½″ × 2½″ × 5/16″	1·47	4·98	0·84	1·76	...
2½″ × 2½″ × ¼″	1·19	4·04	0·68	1·78	0·50
2¼″ × 2¼″ × ⅜″	1·55	5·26	0·65	1·53	...
2¼″ × 2¼″ × 5/16″	1·31	4·45	0·59	1·57	...
2¼″ × 2¼″ × ¼″	1·06	3·61	0·49	1·61	0·46
2″ × 2″ × ⅜″	1·36	4·62	0·42	1·36	...
2″ × 2″ × 5/16″	1·15	3·92	0·39	1·41	...
2″ × 2″ × ¼″	0·94	3·19	0·33	1·46	0·41
2″ × 1½″ × ⅜″	1·17	3·98	0·20	1·04	...
2″ × 1½″ × 5/16″	1·00	3·39	0·18	1·06	...
2″ × 1½″ × ¼″	0·81	2·76	0·15	1·08	0·40
2″ × 1¼″ × ¼″	0·75	2·55	0·09	0·92	...
2″ × 1¼″ × 3/16″	0·57	1·95	0·07	0·94	0·40
1¾″ × 1¾″ × ¼″	0·81	2·76	0·24	1·22	...
1¾″ × 1¾″ × 3/16″	0·62	2·11	0·20	1·23	0·36
1½″ × 2″ × ¼″	0·81	2·76	0·31	1·33	...

Section. Ordinary tees. Table × web.	Area in sq. inches.	Weight in lbs. per foot run.	Greatest moment of inertia about axis a—b. Fig. 11.	Distance of centre of gravity above lower edge.	Approximate least radius of gyration about axis c—d.
$1\frac{1}{2}'' \times 2'' \times \frac{3}{16}''$	0·62	2·11	0·25	1·36	0·29
$1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$	0·69	2·34	0·12	1·03	...
$1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{3}{16}''$	0·53	1·79	0·11	1·05	0·30
$1\frac{1}{4}'' \times 1\frac{1}{4}'' \times \frac{1}{4}''$	0·56	1·90	0·07	0·85	...
$1\frac{1}{4}'' \times 1\frac{1}{4}'' \times \frac{3}{16}''$	0·45	1·53	0·06	0·88	0·24
$1'' \times 1'' \times \frac{3}{16}''$	0·34	1·15	0·03	0·67	...
$1'' \times 1'' \times \frac{1}{8}''$	0·23	0·78	0·02	0·71	0·20

TABLE No. 26.

The Principal Mechanical Elements of Bulb Tees

(See Fig. 12).

Depth × table × thickness of web and table.	Area in sq. inches.	Weight in lbs. per foot lineal.	Greatest moment of inertia about axis $a-b$. Fig. 12.	Distance of axis $a-b$ from farthest edge of section.	Approximate least radius of gyration about axis $c-d$.
$12'' \times 6\frac{1}{2}'' \times \frac{11}{20}''$	11·67	39·68	202·2	6·80	1·07
$11\frac{1}{2}'' \times 6\frac{1}{2}'' \times \frac{10}{20}''$	10·02	34·07	164·4	6·30	1·07
$11'' \times 6'' \times \frac{9}{20}''$	9·07	30·84	151·7	6·07	0·99
$10'' \times 6'' \times \frac{9}{20}''$	8·33	28·33	120·0	5·72	0·99
$9'' \times 5\frac{1}{2}'' \times \frac{9}{20}''$	7·43	25·26	80·2	5·21	0·91
$9'' \times 5\frac{1}{4}'' \times \frac{8}{20}''$	6·70	22·78	76·5	5·06	0·86
$8\frac{1}{2}'' \times 5\frac{1}{4}'' \times \frac{8}{20}''$	6·33	21·52	64·5	4·90	0·86
$8'' \times 5\frac{1}{2}'' \times \frac{8}{20}''$	6·70	22·78	56·3	4·85	0·92
$8'' \times 5'' \times \frac{8}{20}''$	5·91	20·10	51·2	4·67	0·82
$7'' \times 5'' \times \frac{8}{20}''$	5·32	18·10	34·3	4·25	0·82
$6'' \times 4\frac{1}{2}'' \times \frac{8}{20}''$	4·68	15·91	20·4	3·63	0·74
$5'' \times 4'' \times \frac{8}{20}''$	2·46	8·36	10·0	3·16	0·66
$3'' \times 3'' \times \frac{6}{16}''$	1·71	5·81	2·2	1·86	0·49

Rolled Joists.—This well-known, most useful, and deservedly popular section is shown in Fig. 13.

The web is most commonly rolled with parallel sides, the flanges being tapered, and connected to the web with roundings in the internal corners. The proportion of web thickness to flange thickness, the amount of taper on the latter, the radii of the roundings, have been, as in other sections, variable with different

makers. These proportions exert some influence on the precise values of the moments of inertia and resistance, and the manufacturers of this section frequently give in their trade catalogues the mechanical elements and exact proportions of the sections rolled by them. This course is commendable in preference to the compilation of tables of strengths in which the data of the calculations are absent.

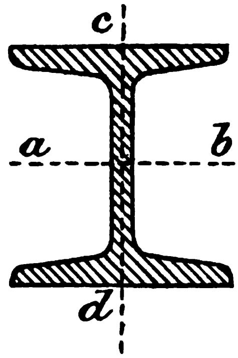


FIG. 13.

In the British standard section the thickness of web and flanges, the taper of the latter, and the radii of the connecting curves, are all standardized, and the mechanical elements of the standard section will be found in the publication previously referred to.

The depths of this section as usually found in the market range from 3 inches to 24 inches, and the width of flange from 1½ inch to 8 inches.

This width of flange has recently been exceeded in continental rolling mills, and the section thus produced offers considerable advantages in column design owing to the increase of the least radius of gyration, and in the arrangement of details in connections, where the additional space afforded is often very convenient. Notwithstanding the width of flange the section can be very cleanly rolled, right out to the edge of the flange, and straight and true in its length.

The proportions of depth and width require careful consideration when selection is being made of a section suitable for the purpose in view. The economy of this section as regards riveting, and the facility with which, aided by the table of strengths obligingly furnished by the manufacturer, the selection of a section for strength can be made, undoubtedly contribute to the favour in which the rolled joist is held. It is questionable, however, as a matter of taste, how far the indiscriminate use of the section, especially in the largest sizes, contributes to the artistic appearance, if it may be so called, of well-designed ironwork, and it must be confessed that economy of both cost in manufacture and painstaking in design are frequently attained at the expense of appearances. It is to be feared, however, that any regard for appearances in structural steelwork, if it implies

any increase in cost, real or imaginary, will in these competitive days be regarded by many as an economic heresy.

No universally recognized standard of proportion of the flanges and web of rolled joists had, up to a recent period, been arrived at by manufacturers. Published lists of sections show considerable variation in the proportion of web thickness to flange width, of web thickness to height of joist, of mean thickness of flange as compared with width, or with height of joist. The thickness of web is found to range between seven and twelve hundredths of the flange width in joists of 3½ inches width of flange and upwards, and may be taken to average about eight hundredths. In joists under 3½ inches in flange width the web will average about one-tenth of flange width. The mean thickness of flange is equally variable, and will be found to range between five and nine hundredths of the height of joist in joists above 6 inches high. In shallower joists the mean flange thickness will range from eight to twelve hundredths.

The maximum moment of inertia of the cross-section will increase in value per unit of area as the web becomes thinner, but the student need not be reminded that the moment of inertia is not the only standard of the ultimate actual strength of the joist. Apart from the practical requirements of the rolling mill the web must be thick enough not only to withstand the usual web stresses, but also to resist the effects of corrosion, and to assist the top flange to resist the buckling tendency under compression which is found in practice to limit the strength of the joist when not supported laterally, the compression flange under these conditions usually failing by lateral flexure before the full tensile resistance of the metal in the lower flange has been attained.

Within the usual limits of variation of web thickness as rolled by different manufacturers, the maximum value of the moment of inertia compared with the total sectional area or weight per foot run will be attained when the flange thickness is from nine to ten hundredths of the height of the girder, but the economic efficiency is practically equally as great between the limits of six and twelve hundredths, and the lower value of flange thickness is the one more usually found in joists above 6 inches in height.

The following table is based upon an assumed web thickness of 0·08 (width of flange) in all joists above 3½ inches wide, and 0·10 (width of flange) in joists under that width. The values of the

moments of inertia (taken about the axis *a—b*) are given for varying proportions of flange thickness, and these will be found to cover the variations usually found in practice. The radii of gyration (for use in column and strut design) are given about the axes *a—b* and *c—d* respectively.

TABLE No. 27.

The Principal Mechanical Elements of Rolled Joists.

(See Fig. 13.)

Flange thickness / Depth of joist.	Section of joist.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis *a—b*. Fig. 13.	Radii of gyration. Axis *a—b*.	Radii of gyration. Axis *c—d*.	Distance of axis *a—b* from farthest edge of section.
0·05	20″ × 7½″	25·8	88	1646·4	7·92	1·60	10·00
0·06	,, ,,	28·56	97	1864·8	8·08	1·72	,,
0·05	19¾″ × 7¼″	24·59	84	1530·0	7·85	1·60	9·87
0·06	,, ,,	27·25	93	1735·8	7·99	1·66	,,
.0·05	18″ × 7″	21·67	74	1120·2	7·20	1·50	9·00
0·06	,, ,,	23·99	81	1268·8	7·27	1·60	,,
0·05	17¾″ × 6¾″	20·57	70	1029·2	7·09	1·49	8·87
0·06	,, ,,	22·80	78	1165·7	7·11	1·55	,,
0·05	16″ × 6″	16·51	56	674·3	6·38	1·22	8·00
0·06	,, ,,	18·27	62	763·8	6·40	1·25	,,
0·05	16″ × 5″	13·76	46	561·9	6·42	1·10	8·00
0·06	,, ,,	15·23	52	636·5	6·46	1·14	,,
0·05	15¾″ × 6⅛″	16·55	56	656·4	6·30	1·35	7·87
0·06	,, ,,	18·35	62	743·7	6·37	1·41	,,
0·05	15″ × 6″	15·48	53	555·6	5·98	1·30	7·50
0·07	,, ,,	18·79	64	700·0	6·10	1·35	,,
0·05	15″ × 5½″	14·19	48	509·3	5·97	1·21	7·50
0·07	,, ,,	17·22	58	641·7	6·10	1·26	,,
0·05	15″ × 5″	12·90	44	463·0	5·98	1·00	7·50
0·06	,, ,,	14·28	49	524·5	6·05	1·04	,,
0·05	14″ × 6″	14·45	49	451·8	5·62	1·30	7·00
0·06	,, ,,	15·99	54	511·7	5·65	1·35	,,
0·05	14″ × 5½″	13·24	45	414·1	5·60	1·21	7·00
0·06	,, ,,	14·66	50	469·0	5·64	1·26	,,
0·05	13″ × 5″	11·18	38	301·4	5·18	1·10	6·50
0·06	,, ,,	12·37	42	341·4	5·24	1·14	,,
0·06	12″ × 6½″	14·85	51	349·1	4·84	1·49	6·00
0·07	,, ,,	16·29	55	388·3	4·88	1·54	,,
0·06	12″ × 6″	13·71	47	322·2	4·85	1·35	6·00
0·08	,, ,,	16·36	56	392·8	4·92	1·40	,,

$\frac{\text{Flange thickness}}{\text{Depth of joist.}}$	Section of joist.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis a—b. Fig. 13.	Radii of gyration.		Distance of axis a—b from farthest edge of section.
					Axis a—b.	Axis c—d.	
0·06	12″ × 5½″	12·56	43	295·4	4·86	1·26	6·00
0·08	,, ,,	14·99	51	359·9	4·90	1·33	,,
0·05	12″ × 5″	10·32	35	237·1	4·80	1·10	6·00
0·06	,, ,,	11·42	38½	268·5	4·85	1·15	,,
0·07	10½″ × 5″	10·96	37	200·1	4·27	1·18	5·25
0·08	,, ,,	11·93	41	219·3	4·29	1·21	,,
0·05	10¼″ × 4¾″	8·37	28½	140·3	4·08	1·04	5·125
0·06	,, ,,	9·25	31½	159·0	4·17	1·09	,,
0·06	10″ × 6″	11·42	38½	186·5	4·04	1·38	5·00
0·08	,, ,,	13·63	46·5	227·3	4·08	1·46	,,
0·05	10″ × 5″	8·60	29	137·2	3·99	1·10	5·00
0·08	,, ,,	11·36	38½	189·4	4·08	1·21	,,
0·06	10″ × 4½″	8·57	29	139·8	4·03	1·03	5·00
0·08	,, ,,	10·22	35	170·5	4·08	1·09	,,
0·06	9½″ × 4½″	8·12	27½	119·6	3·88	1·03	4·75
0·08	,, ,,	9·70	33	145·8	3·90	1·09	,,
0·06	9¼″ × 4″	7·04	24	98·4	3·74	0·91	4·625
0·08	,, ,,	8·40	28½	119·9	3·78	0·96	,,
0·07	9″ × 7″	13·15	44½	176·4	3·66	1·65	4·50
0·09	,, ,,	15·47	52½	209·5	3·68	1·73	,,
0·06	9″ × 5½″	9·40	32	124·6	3·64	1·26	4·50
0·08	,, ,,	11·24	38	151·9	3·68	1·33	,,
0·06	9″ × 4″	6·84	23¼	90·6	3·64	0·92	4·50
0·08	,, ,,	8·18	28	110·5	3·67	1·08	,,
0·06	9″ × 3″	5·61	19	70·4	3·55	0·66	4·50
0·08	,, ,,	6·59	22½	85·0	3·59	0·70	,,
0·06	8¼″ × 3″	5·15	17¼	54·3	3·25	0·66	4·125
0·08	,, ,,	6·04	20½	65·5	3·29	0·70	,,
0·06	8″ × 6″	9·14	31	95·5	3·23	1·37	4·00
0·08	,, ,,	10·90	37	116·4	3·27	1·45	,,
0·06	8″ × 5″	7·61	26	79·5	3·23	1·15	4·00
0·08	,, ,,	9·09	31	97·0	3·27	1·21	,,
0·06	8″ × 4″	6·09	20¾	63·6	3·23	0·92	4·00
0·08	,, ,,	7·27	24¾	77·6	3·27	0·97	,,
0·06	7″ × 4″	5·33	18	42·64	2·83	0·92	3·50
0·08	,, ,,	6·36	21½	51·98	2·86	1·08	,,
0·05	7″ × 3¾″	4·51	15¼	35·29	2·80	0·83	3·50
0·06	,, ,,	5·00	17	39·97	2·83	0·86	,,
0·06	6¼″ × 3½″	4·15	14	26·56	2·53	0·80	3·125
0·08	,, ,,	4·96	17	32·37	2·56	0·85	,,
0·06	6¼″ × 3″	3·88	13¼	23·60	2·47	0·66	3·125
0·08	,, ,,	4·57	15½	28·47	2·50	0·70	,,
0·06	6¼″ × 2″	2·60	8¾	15·73	2·46	0·44	3·125

Flange thickness / Depth of joist.	Section of joist.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis a—b. Fig. 13.	Radii of gyration.		Distance of axis a—b from farthest edge of section.
					Axis a—b.	Axis c—d.	
0·08	$6\frac{1}{4}'' \times 2''$	3·05	$10\frac{1}{4}$	18·98	2·49	0·47	3·125
0·07	$6'' \times 5''$	6·26	$21\frac{1}{4}$	37·33	2·45	1·18	3·00
0·09	,, ,,	7·37	25	44·34	2·45	1·24	,,
0·06	$6'' \times 4\frac{1}{2}''$	5·14	$17\frac{1}{4}$	30·21	2·43	1·03	3·00
0·08	,, ,,	6·13	21	36·83	2·46	1·09	,,
0·06	$6'' \times 3''$	3·74	$12\frac{3}{4}$	20·87	2·37	0·66	3·00
0·08	,, ,,	4·39	15	25·19	2·39	0·70	,,
0·06	$6'' \times 2''$	2·49	$8\frac{1}{2}$	13·92	2·36	0·44	3·00
0·08	,, ,,	2·93	10	16·79	2·39	0·47	,,
0 06	$5\frac{1}{2}'' \times 2''$	2·29	$7\frac{3}{4}$	10·72	2·16	0·44	2·75
0·08	,, ,,	2·68	$9\frac{1}{4}$	12·94	2·19	0·47	,,
0·06	$5\frac{1}{4}'' \times 1\frac{1}{2}''$	1·63	$5\frac{1}{2}$	6·99	2·07	0·33	2·625
0·08	,, ,,	1·92	$6\frac{1}{2}$	8·44	2·09	0·36	,,
0·10	$5'' \times 5''$	6·60	$22\frac{1}{2}$	27·55	2·04	1·25	2·50
0·12	,, ,,	7·52	$25\frac{1}{2}$	31·05	2·04	1·29	,,
0·10	$5'' \times 4\frac{1}{2}''$	5·94	$20\frac{1}{4}$	24·79	2·04	1·13	2·50
0·12	,, ,,	6·75	23	27·94	2·04	1·16	,,
0·08	$5'' \times 4\frac{1}{4}''$	4·83	$16\frac{1}{4}$	20·13	2·04	1·03	2·50
0·10	,, ,,	5·61	19	23·42	2·04	1·07	,,
0·08	$5'' \times 3''$	3·66	$12\frac{1}{2}$	14·58	2·00	0·70	2·50
0·10	,, ,,	4·20	$14\frac{1}{4}$	16·85	2·00	0·73	,,
0·08	$4\frac{3}{4}'' \times 1\frac{3}{4}''$	2·03	$6\frac{3}{4}$	7·29	1·90	0·41	2·375
0·10	,, ,,	2·33	8	8·42	1·90	0·44	,,
0·08	$4'' \times 3''$	2·93	10	7·46	1·60	0·70	2·00
0·10	,, ,,	3·36	$11\frac{1}{2}$	8·62	1·60	0·73	,,
0·07	$4'' \times 2''$	1·81	$6\frac{1}{4}$	4·50	1·58	0·45	2·00
0·09	,, ,,	2·09	$7\frac{1}{4}$	5·37	1·60	0·48	,,
0·06	$4'' \times 1\frac{3}{4}''$	1·45	5	3·61	1·58	0·38	2·00
0·09	,, ,,	1·83	$6\frac{1}{2}$	4·70	1·60	0·42	,,
0·08	$3\frac{1}{2}'' \times 3''$	2·56	$8\frac{3}{4}$	5·00	1·39	0·70	1·75
0·10	,, ,,	2·94	10	5·78	1·40	0·73	,,
0·08	$3\frac{1}{2}'' \times 1\frac{1}{2}''$	1·28	$4\frac{1}{2}$	2·50	1·39	0·35	1·75
0·10	,, ,,	1·47	5	2·89	1·40	0·37	,,
0·11	$3'' \times 3''$	2·68	$9\frac{1}{4}$	3·86	1·19	0·74	1·50
0·13	,, ,,	3·00	$10\frac{1}{4}$	4·29	1·19	0·76	,,
0·08	$3'' \times 1\frac{1}{2}''$	1·09	$3\frac{1}{2}$	1·57	1·20	0·35	1·50
0·10	,, ,,	1·26	$4\frac{1}{4}$	1·82	1·20	0·37	,,
0·08	$3'' \times 1\frac{1}{4}''$	0·91	3	1·31	1·20	0·29	1·50
0·10	,, ,,	1·05	$3\frac{1}{2}$	1·51	1·20	0·30	,,

It is customary to specify the width of flange and total depth coupled with the weight per foot lineal of the rolled joist required,

and this is doubtless the most desirable course to pursue. It leaves, however, the exact relative thicknesses of web and flanges an open question, though the total sectional area is of course governed by the weight per foot. If, on the other hand, the designer specifies the thickness of web or flange, he must in all probability be prepared to accept the section rolled by some one particular maker, and in such a case he will do well to follow the dimensions given in the trade section books. These remarks do not of course apply to the use of the British standard section, where the thicknesses of web and flanges for the given depth, width, and weight are standardized. As regards the values of the moment of inertia given in the above table, and based upon the proportions of web and flange stated, it may be remarked that for any weight per foot lineal of joist of any one particular section not found in the table, the moment of inertia for that weight may for approximate calculations be taken as simply proportional to the weight per foot.

The value of the least radius of gyration will not be found to vary materially for any practical variation of the section within the limits usually rolled.

Channels.—This section is represented in Fig. 14. The web is rolled with parallel sides, the flanges are tapered and connected to the web with rounded internal angles, and this is the type of the British standard section. Increase of weight beyond the minimum section is obtained mainly by an increase in web thickness.

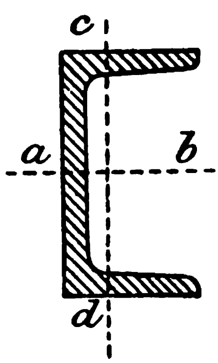


Fig. 14.

This section is frequently used as a beam in small bearers, as a strut in the compression members of lattice girders and roof trusses, and in riveted columns, while it is occasionally useful in certain connections as taking the place of two angles.

If a small section of channel is required having a rivet through the web, as, for example, in the case of two channels crossing one another, back to back, and riveted together, care must be taken in the selection to secure one wide enough to permit of the formation of the point of the rivet. For this reason in such cases a small angle will frequently be found preferable to a small channel.

As in rolled joist sections so in channels, no generally recognized standard of proportionate thickness in web or flanges appears to have been attained prior to the establishment of the British standard section. In the majority of cases the thickness of web and flanges is similar or nearly so, but occasional sections are found where the thickness of flange is greater than that of the web. Any increase in weight over the normal or minimum section is obtained by thickening the web, the flange thickness remaining practically constant.

In the following table the thickness of web and flanges is assumed as uniform all over. The standard thickness of the British standard channel shows a flange thickness greater than that of the web, and the student is referred to the "Properties of British Standard Sections" for the corresponding mechanical values.

TABLE No. 28.

The Principal Mechanical Elements of Channels.

(See Fig. 14.)

Section of channel.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis a—b. Fig. 14.	Radii of gyration.		Distance of axis a—b from farthest edge of section.
				Axis a—b.	Axis c—d.	
$15\frac{3}{4}'' \times 3'' \times \frac{1}{2}''$	10·37	$35\frac{1}{4}$	308·2	5·45	0·74	7·87
$15'' \times 4'' \times \frac{5}{8}''$	13·59	$46\frac{1}{4}$	393·8	5·38	1·01	7·50
$12'' \times 4'' \times \frac{1}{2}''$	9·50	$32\frac{1}{4}$	187·8	4·45	1·14	6·00
$12'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	9·00	$30\frac{1}{2}$	171·2	4·36	0·97	6·00
$12'' \times 3'' \times \frac{7}{16}''$	7·48	$25\frac{1}{2}$	138·5	4·30	0·81	6·00
$12'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	8·00	$27\frac{1}{4}$	138·1	4·15	0·62	6·00
$11\frac{7}{8}'' \times 3'' \times \frac{1}{2}''$	8·43	$28\frac{3}{4}$	150·5	4·22	0·81	5·93
$10'' \times 4'' \times \frac{7}{16}''$	7·50	$25\frac{1}{2}$	107·8	3·79	1·17	5·00
$10'' \times 3'' \times \frac{7}{16}''$	6·62	$22\frac{1}{2}$	87·8	3·64	0·83	5·00
$10'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$	6·18	21	77·8	3·55	0·66	5·00
$9\frac{7}{8}'' \times 3\frac{1}{8}'' \times \frac{7}{16}''$	6·66	$22\frac{3}{4}$	87·5	3·62	0·88	4·94
$9\frac{1}{4}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	6·72	$22\frac{3}{4}$	81·0	3·47	1·00	4·62
$9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	6·62	$22\frac{1}{2}$	75·7	3·38	1·00	4·50
$9'' \times 3'' \times \frac{3}{8}''$	5·34	$18\frac{1}{4}$	59·4	3·33	0·86	4·50
$9'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	4·96	17	52·4	3·25	0·68	4·50
$8\frac{1}{4}'' \times 2\frac{1}{2}'' \times \frac{7}{16}''$	5·41	$18\frac{1}{2}$	48·0	2·98	0·68	4·12
$8'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	7·00	$23\frac{3}{4}$	63·58	3·10	1·00	4·00
$8'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$	6·00	$20\frac{1}{2}$	49·50	2·87	0·68	4·00
$8'' \times 2\frac{1}{4}'' \times \frac{3}{8}''$	4·40	15	36·45	2·88	0·61	4·00

Section of channel.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis *a—b*. Fig. 14.	Radii of gyration.		Distance of axis *a—b* from farthest edge of section.
				Axis *a—b*.	Axis *c—d*.	
$8'' \times 2'' \times \frac{3}{8}''$	4·22	$14\frac{1}{2}$	33·73	2·83	0·51	4·00
$7\frac{7}{8}'' \times 3\frac{3}{4}'' \times \frac{1}{2}''$	7·18	$24\frac{1}{2}$	64·60	3·00	1·13	3·94
$7\frac{7}{8}'' \times 3\frac{1}{8}'' \times \frac{1}{2}''$	6·56	$22\frac{1}{4}$	56·10	2·92	0·91	3·94
$7\frac{7}{8}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	4·55	$15\frac{1}{2}$	37·69	2·88	0·68	3·94
$7'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	6·50	$22\frac{1}{4}$	46·04	2·66	1·10	3·50
$7'' \times 3'' \times \frac{1}{2}''$	6·00	$20\frac{1}{2}$	40·75	2·61	0·88	3·50
$7'' \times 2'' \times \frac{3}{8}''$	3·84	13	24·10	2·51	0·53	3·50
$6'' \times 4'' \times \frac{1}{2}''$	6·50	$22\frac{1}{4}$	35·54	2·34	1·24	3·00
$6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	4·59	$15\frac{3}{4}$	25·31	2·34	1·01	3·00
$6'' \times 3'' \times \frac{3}{8}''$	4·22	$14\frac{1}{2}$	22·34	2·30	0·91	3·00
$6'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	3·84	$13\frac{1}{4}$	19·37	2·24	0·73	3·00
$6'' \times 2'' \times \frac{3}{8}''$	3·47	12	16·40	2·17	0·55	3·00
$6'' \times 1\frac{3}{4}'' \times \frac{5}{16}''$	2·78	$9\frac{1}{2}$	12·89	2·15	0·42	3·00
$5\frac{3}{4}'' \times 2\frac{3}{8}'' \times \frac{1}{2}''$	4·75	$16\frac{1}{4}$	20·87	2·10	0·71	2·87
$5\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	3·65	$12\frac{1}{2}$	15·68	2·07	0·74	2·75
$5\frac{1}{8}'' \times 2\frac{7}{8}'' \times \frac{1}{2}''$	4·94	17	18·32	1·92	0·86	2·56
$5'' \times 2'' \times \frac{3}{8}''$	3·09	$10\frac{1}{2}$	10·43	1·84	0·58	2·50
$5'' \times 1\frac{1}{2}'' \times \frac{3}{8}''$	2·72	$9\frac{1}{4}$	8·42	1·76	0·40	2·50
$4\frac{1}{2}'' \times 2'' \times \frac{3}{8}''$	2·90	10	8·05	1·66	0·58	2·25
$4\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{3}{8}''$	2·53	$8\frac{1}{2}$	6·45	1·60	0·41	2·25
$4\frac{1}{8}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$	3·14	$10\frac{3}{4}$	7·81	1·57	0·75	2·06
$4'' \times 1\frac{3}{4}'' \times \frac{5}{16}''$	2·14	$7\frac{1}{4}$	4·73	1·49	0·42	2·00
$4'' \times 1\frac{1}{2}'' \times \frac{5}{16}''$	2·00	7	4·19	1·45	0·41	2·00
$3\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{3}{8}''$	2·15	$7\frac{1}{4}$	3·42	1·26	0·43	1·75
$3'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$	1·37	$4\frac{3}{4}$	1·75	1·13	0·45	1·50
$2\frac{3}{8}'' \times 1\frac{3}{16}'' \times \frac{1}{4}''$	1·06	$3\frac{3}{4}$	0·81	0·87	0·35	1·18
$2\frac{1}{4}'' \times 1\frac{1}{4}'' \times \frac{1}{4}''$	1·06	$3\frac{3}{4}$	0·74	0·83	0·38	1·12
$2'' \times 1\frac{1}{8}'' \times \frac{1}{4}''$	0·94	$3\frac{1}{4}$	0·50	0·73	0·33	1·00
$1\frac{3}{4}'' \times \frac{5}{8}'' \times \frac{3}{16}''$	0·49	$1\frac{3}{4}$	0·18	0·61	0·18	0·87
$1\frac{1}{2}'' \times 1\frac{1}{4}'' \times \frac{3}{16}''$	0·68	$2\frac{1}{2}$	0·23	0·58	0·38	0·75
$1\frac{1}{2}'' \times \frac{3}{4}'' \times \frac{3}{16}''$	0·49	$1\frac{3}{4}$	0·14	0·53	0·22	0·75

Further reference will be made to the use of channels in the practical design of columns or struts.

Zed Angles.—This useful section is shown in Fig. 15. It is largely used in the frames of ship and caisson work, having a considerable moment of inertia for its weight, as compared with angles or tees, with ample width of flange for riveted connections.

The web is rolled with parallel sides, the flanges having a taper and being connected to the web by curves at the internal

angles. In the British standard section the flanges have no taper, but are of uniform thickness.

Increase of weight beyond the minimum section is obtained by thickening the web, the width of flange being slightly increased.

The section is frequently rolled with a uniform thickness of web and flange, the latter being tapered as above described, and the quoted thickness being the mean between that of the root and of the point of the flange. The flanges of the British standard section have a thickness in excess of that of the web.

Occasionally the flanges are rolled of unequal width; this is a convenience where additional width is required for heavy riveting, and in those cases where the lesser width of flange is sufficient for the riveted attachments, then the increased width of the other flange yields a larger moment of inertia. The thicknesses given in the following table are approximately those to which the various sections are rolled as a minimum; for other thicknesses than those given, the moment of inertia may be taken for approximate calculations as proportional to the sectional area or weight for each section.

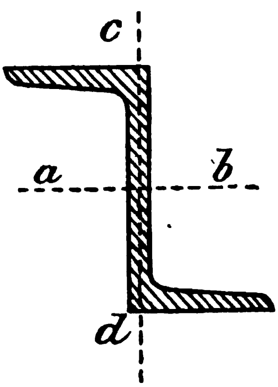


FIG. 15.

The table on page 104 gives the dimensions, sectional area, weight per foot lineal, moments of inertia, radii of gyration, etc., for Zed angles.

Further reference will be made to the use of Zed angles in the practical design of columns or struts.

In the preceding pages the sections which have been described and of which the tables of the principal mechanical elements have been given, viz. angles, equal and unequal-legged, tees, bulb tees or deck beams, rolled joists, channels, and Zed angles, are those which may be called the elementary or standard sections, which in combination with plates and bars are most ordinarily employed in riveted constructional steelwork. It is not possible to consider in detail the very numerous forms of rolled sections, other than those above mentioned, employed for special purposes. These include, for example, the varied sections of railway bars (bull-headed, bridge, and flat-footed), fish plates, tramway rails, guard

TABLE No. 29.

THE PRINCIPAL MECHANICAL ELEMENTS OF ZED ANGLES.
(See Fig. 15.)

Section of Zed angle. Depth × flanges.	Area in square inches.	Weight in lbs. per foot run.	Moment of inertia about the axis *a—b*. Fig. 15.	Radii of gyration.		Distance of axis *a—b* from farthest edge of section.
				Axis *a—b*.	Axis *c—d*.	
10″ × 3½″ × 3½″ × ½″	8·00	27¼	109·41	3·70	1·20	5·00
8″ × 3½″ × 3½″ × ½″	7·00	23¾	63·58	3·10	1·28	4·00
7″ × 3½″ × 3½″ × ½″	6·50	22¼	46·04	2·66	1·33	3·50
7″ × 3″ × 3″ × ⅜″	4·59	15¾	32·34	2·65	1·13	3·50
6″ × 3½″ × 3½″ × ⅜″	4·59	15¾	25·31	2·34	1·41	3·00
6″ × 3″ × 3″ × ⅜″	4·22	14½	22·34	2·30	1·15	3·00
5½″ × 3½ × 3½″ × ⅜″	4·40	15	20·61	2·16	1·43	2·75
5½″ × 3″ × 3″ × ⅜″	4·03	13¾	18·15	2·12	1·17	2·75
5″ × 3½″ × 3½″ × ⅜″	4·22	14½	16·47	1·95	1·46	2·50
5″ × 3″ × 3″ × ⅜″	3·84	13	14·46	1·94	1·20	2·50
4″ × 3″ × 3″ × ⅜″	3·47	11¾	8·49	1·56	1·26	2·00

rails, sections of trough flooring (usually formed in hydraulic presses), quadrant sections for pile-work, half-round, segmental, or cope steels, sash-bars, and fancy and other sections.

The mechanical elements of square, round, hexagonal, and octagonal bars have not been given in the tables, as these can be easily obtained by the usual arithmetical processes.

With respect to the use of plates and bars, it is sufficient to point out that the dimensions to which these can now be rolled are amply sufficient to meet all legitimate demands of the designer of constructional steelwork. Various makers have their own standard maximum dimensions to which plates, sheets, or flats can be rolled, and it is customary to assign a limit of superficial area for each thickness of plate, which is not exceeded without entering into special arrangements. Thus for a ⅜-inch plate, the limit of area is given by one authority as 135 square feet, the maximum length of plate being 42 feet, and the maximum width 7 feet 6 inches, it being understood that maximum length and maximum width are not rolled together, but that, given the length, the width is such as not to exceed the limit of area, or *vice versâ*. Again, for a plate ⅞ inch thick, a limit of 250 square feet is given,

the maxima of length and width being 56 feet and 10 feet respectively.

With respect to flats, usually so called when the width does not exceed 12 inches to 15 inches, the available length obtainable without joint will usually be found to meet all practical requirements, as other considerations, such as the maximum length permissible for transport or shipment, very frequently rule the case.

CHAPTER III.

UPON CERTAIN APPLICATIONS OF RIVETED GIRDERWORK, WITH SOME REMARKS UPON RIVETS AND RIVET-HOLES.

General remarks—Examples of various types of girderwork—Remarks upon the design of riveted connections—Fundamental rules and the study of good examples—The making of rivet-holes—Punching and the punching machine—Burrs, and the holes which they imply—Drilled holes—The templet system—Making and use of templets—Combined punched and drilled or rimered holes—Rivets—Shape and dimensions of rivet-heads—Pan-heads—Cup-heads—Percentage of weight of heads and points—Table of weights of heads and points—Methods of riveting—Hand riveting—Hydraulic riveting—Pneumatic riveting—The pneumatic hand hammer and its applications—Girderwork as applied to bridge construction—Example of viaduct construction—Cast-iron cylinders—Details—Lengths of cylinders—Bottom lengths—Upper lengths and cap—Holding-down bolts of main girders—Cylinder bracing—Main girders—Footway and flooring—Cross girders—Expansion arrangements—Roadway—Details in connection with mixed traffic—Curbing—Girderwork for machine or boiler shops, steel foundries, engine-houses, etc.—Traveller girders—Travelling cranes and their loads—Wheel pressures—Crane wheels—Table of weights of overhead travelling cranes—Analysis of total loads and resulting reactions of supports—Minimum dimensions and clearances for overhead travelling cranes—Headway required—Truth of gauge of road for overhead travelling cranes—Types of girders for roadway—Sections of rails and methods of connection—Roadway at walls of shops—Details—Lattice girderwork for roofing—Example and details of riveted connections—Application of girderwork to the support of cast-iron water-tanks—Consideration of the details of the tanks themselves—General arrangements of such tanks—Bottom and side plates—Subdivision of tanks—Plate flanges—Tie rods—Arrangement of girderwork—Details of roofing arrangements in connection with tanks—Gutters and gangways—Connections of pipe-work, etc.—Table of the weight of mild steel bolts and nuts.

RIVETED girderwork in general covers so wide an area of constructive practice, and its application is found in so many different directions, that it is hopeless to deal adequately with the subject in one chapter of such a collection of notes as the present. All that can here be done is to offer to the student some examples of the application of girderwork in one or two particular directions,

accompanied by a few remarks on some practical aspects of rivets and rivet-holes.

Nor can the theory of the beam be in any way entered on. The methods of determining bending moments, either by graphic or analytic methods, the theory of shearing forces, and of stresses in triangulated or latticed structures, together with the processes of apportioning the sectional areas of metal required, and of determining the correct lengths of flange-plates, etc., must be assumed to have been acquired in greater or less degree by the reader of these notes.

The same remark must also be taken to apply to what may be called the theory of riveted joints in the application of safe limits of shear and bearing stresses.[1]

The examples of various types of girderwork which are given in the illustrations which follow are all of comparatively small span, not exceeding 60 feet, as the consideration of girders of very large span does not enter within the limits of these notes.

Thus in Figs. 66 and 73 we have examples of ordinary single-plate web-girders to carry traveller rails, while in Figs. 244 and 245 we have details of a double-webbed or box-girder for the same purpose.

Figs. 116 and 117 show details of single-webbed plate-girders carrying tank-work above, forming a portion of the roof over an engine-house, and it will be convenient to consider, in connection with these girders, such details of the tanks themselves as will be found practically useful to the draughtsman.

In Figs. 33 and 51 are given details of single-web plate-girderwork for bridge construction of the type described, and in Figs. 360 and 361 are found details of box and single-web girders used in jetty construction.

Figs. 82 to 101 show some details of lattice girders for roof construction, especially those details of riveted connections which are all important in these as in other branches of girderwork.

It is somewhat difficult to describe in so many words *all* the mental processes which attend the design by an experienced draughtsman of a well-thought-out riveted connection, and yet

[1] Among the numerous treatises which have been issued on these subjects the student may profitably consult Part IV. of "Notes on Building Construction," in addition to more advanced works on the same subjects; "Bridge Construction," by Prof. T. Claxton Fidler (Griffin); and as regards riveted joints, Prof. Unwin's "Machine Design" (Longmans).

there is no detail associated with the design of structural steel-work which will more reveal the efficiency or otherwise of the designer than this.

It is true that all the mechanical elements which form the basis of the design of the connection may be present to his mind—the total stress, the number and area of rivets, the bearing areas, may all have been correctly determined and provided for, but there will often remain a residuum of conditions to be met outside theoretical requirements as to which there may be a right way or a wrong way of procedure. The experienced draughtsman will almost instinctively choose the right way, although he might find a difficulty in explaining in a few words the reasons for his choice.

Oblique connections of all sorts will generally tax the ingenuity of the draughtsman more than those which are square, and if the conditions on one side of the girder or the connection differ in some way from those on the other, it is always desirable to remember *both ends of the rivet,* and not that end only which is represented on the plane of the paper. There are not wanting in public places evidences of the want of this precaution, which may serve as examples to the junior draughtsman of a wrong method of procedure, of how *not* to design a riveted connection.

The fundamental rules which, after the proper determination of the mechanical elements of the strength of the joint has been made, will govern the general design are: the grouping of the assemblage of rivets on the centre line or line of action of force of the connected members; the reduction to a minimum of loss of section; the proper pitch of rivets, which for certain classes of joints sometimes requires to be as close as possible to avoid clumsiness;[1] the minimum distance from edge of rivet-hole to edge of member to avoid any risk of bursting out; and, lastly, the accessibility of all parts and the provision of sufficient space under all conditions for the operations of riveting and holding up. These rules, if carefully followed and intelligently applied, should lead to satisfactory design. But the study of good examples will be more instructive to the young draughtsman than verbal instructions, however complete, and it is hoped that some assistance will be

[1] In water-tight work and for boilers or receivers rivet spacing is governed by practical consideration of caulking, and the necessity for a sound water or steam or air-tight joint.

derived from the examples figured in the following pages, where the riveting is distinctly shown, although the general scale of the construction may be but small.

The student who, in the course of his inspection of the methods and practice of a girder-maker's or bridge-building yard, observes the process of work carried on by the punching machines, will find one of the results of that operation to consist of a heap, under the

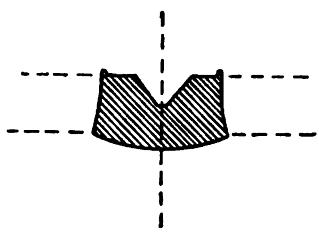


FIG. 16.

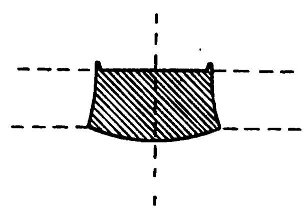


FIG. 17.

machine, of punchings, or, as they are usually called, "burrs,"[1] being the circular discs of metal forced out of the plate or bar in the operation of forming a "punched" rivet-hole. The precise shape and size of the burrs will vary with the diameter of rivet to be employed and the thickness of the material through which the hole is made, but in general will exhibit the features shown in Figs. 16, 17, which are ¾ full-size sections through burrs from punched holes intended for a ¾-inch rivet through a ⅝-inch plate or bar, the precise features of the upper and under surfaces of the burr varying with the type of punch employed, the flow of material prior to, or simultaneous with, the final shearing of the circumferential area being shown by the bulging of the under surface of the burr as shown.

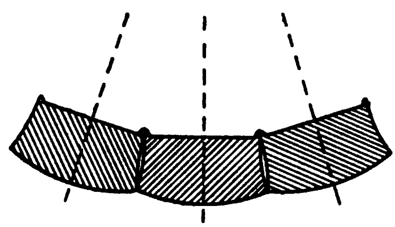


FIG. 18.

If the student now lays, say, three of these burrs together, so that their circumferences are in close contact, as shown in Fig. 18, it at once becomes evident that the burrs are not portions of

[1] For the use of "burrs" in the formation of special concrete in ballast of maximum density, see p. 402.

cylinders, but portions of cones, the angle of the cone being determined by the amount of clearance between the punch and the die, the greater the clearance the greater being the departure from a truly cylindrical form. Thus, as the burr is conical, the hole in the plate or bar is also conical, and we arrive at one of the distinctive features of a punched hole.

Callipered measurements from burrs will show an average difference of about $\frac{1}{16}$ inch between the larger and smaller diameters of the burr, being equivalent to a rate of slope in the side of the cone of about 1 in 8, in plates of from $\frac{3}{8}$ inch to $\frac{1}{2}$ inch thick.

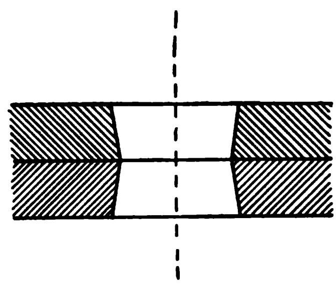


Fig. 19.

It is true that in those cases where two bars or plates are to be riveted together (and two only), as shown in Fig. 19, and the holes have been punched from the meeting or "faying" surfaces, the double cone produces an approximation to a double countersink, and is not so far objectionable, as it possesses in itself a certain element of

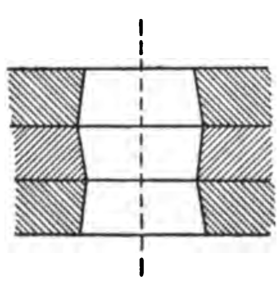


Fig. 20.

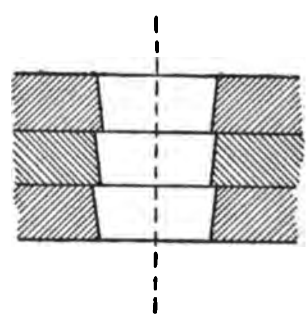


Fig. 21.

strength to resist pulling apart of the plates, even if the heads and points were absent.

But where the number of thicknesses to be riveted together exceeds two, we have a condition of affairs which may assume a variety of shapes according to circumstances, as sketched in Figs. 20, 21, although by a use of the conical drift, which can hardly be called legitimate, some approximation to a roughly

cylindrical hole may be obtained at the cost of a considerable amount of rough usage and distress of the surrounding metal. Thus far we have assumed the axis of the conical holes to be perfectly straight, or, in other words, that the holes have been truly centred one over the other in all the thicknesses passed through. If this be not so, the conditions become aggravated, the quality of the work deteriorates in the same degree, while the illegitimate use of the drift becomes still more pronounced.

The ideal rivet-hole is truly cylindrical, each hole in each thickness of plate or bar being exactly concentric with the adjacent holes, so that the axis of the cylinder remains perfectly straight and square to the plane of junction, whatever be the number of thicknesses joined. These conditions are only perfectly attained when the holes are drilled through all the thicknesses of plates at one operation, and this method is frequently adopted in special cases, or where the conditions or importance of the work render such a course desirable, multiple drills being sometimes employed, by which the position of a number of holes can be simultaneously and very accurately determined with respect to each other. But for the ordinary run of structural steelwork with which we are here mainly concerned, a process such as this is found costly or inconvenient, and other means must be adopted to secure not only that the holes in separate plates shall be truly concentric when assembled together, but also that their pitch or position with respect to each other shall be accurate.

In punched work the holes in each individual plate or bar are punched separately, and this is also the case with drilled work, except in the special cases above mentioned. It is, therefore, in the assembling of these separate parts together prior to the insertion of the rivet that the accuracy or want of accuracy of the methods adopted becomes evident, and the examination of the holes for rivets or other connections becomes an important part of the duty of those who may be charged with the inspection of riveted steelwork.

In the bulk of structural steelwork of good quality the method by which the accuracy of the setting out of rivet-holes is maintained is that known as the "templet" system, and the "templet shop" in a bridge-building or girder-building yard occupies an important position, inasmuch as the care with which the work is set out in this shop is a very powerful factor in the ultimate quality of the finished work. The templets employed are

sometimes of iron, but generally of wood, and the setting out of templet work may be described as careful full-size draughtsmanship on wood, each rivet-hole being accurately set out in its correct position, whether it be a hole near the edge of a plate, a hole in an angle cover, or in any other position required, and bored through the thickness of the templet, to suit the size of a centre punch, which, being passed through the hole in the templet with a blow from the hammer in the hand of the plater, marks, as shown in Fig. 22, upon the surface of the steel plate or bar the centre of the rivet-hole which is subsequently punched or drilled out.

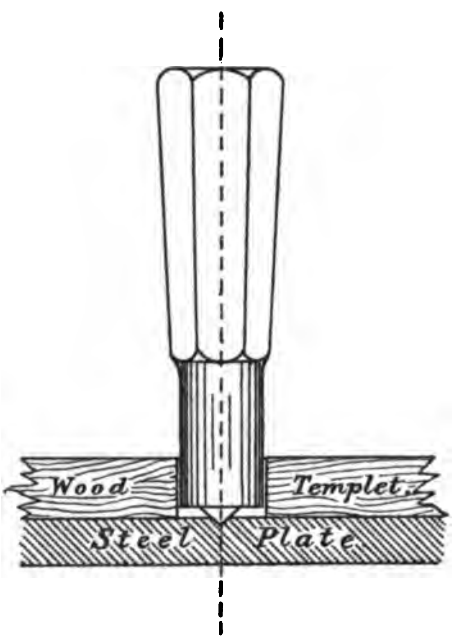


FIG. 22.

The next stage of the process is one which at first sight appears to offer opportunity of error which would go far to destroy the original accuracy of the templet, for as the bar or plate (frequently of considerable dimensions and weight) is passed through the punching machine, the operation of placing the centre-punch mark exactly under the centre of the punch demands skill of eye and hand (assisted sometimes by certain mechanical devices, such as racks, etc.) on the part of the mechanic in charge of the machine. A very considerable degree of accuracy may nevertheless be attained, as is proved when good work of this class is assembled together.

Excellent work can be produced by a system which stands intermediate between punched and drilled work and partakes of some of the advantages of both. Each rivet-hole is first punched out, the largest diameter of the punched hole being from $\frac{1}{16}$ inch to $\frac{1}{8}$ inch less than the diameter of the finished drilled hole. The plate or bar is then transferred from the punching machine to the

drilling machine, and the punched hole is enlarged, or rimered, to the finished diameter required, as shown in Fig. 23. It follows that the conical hole has disappeared, together with a certain zone of metal which may have been overstrained or distressed in the process of punching, and is replaced by a truly cylindrical hole. As, however, the point of the drill, or rimer, in entering the punched hole is guided in direction by that hole, the axes of the punched and drilled holes remain the same for all practical

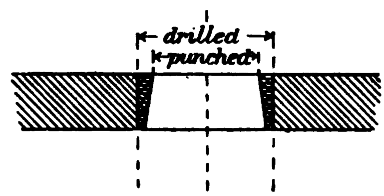


FIG. 23.

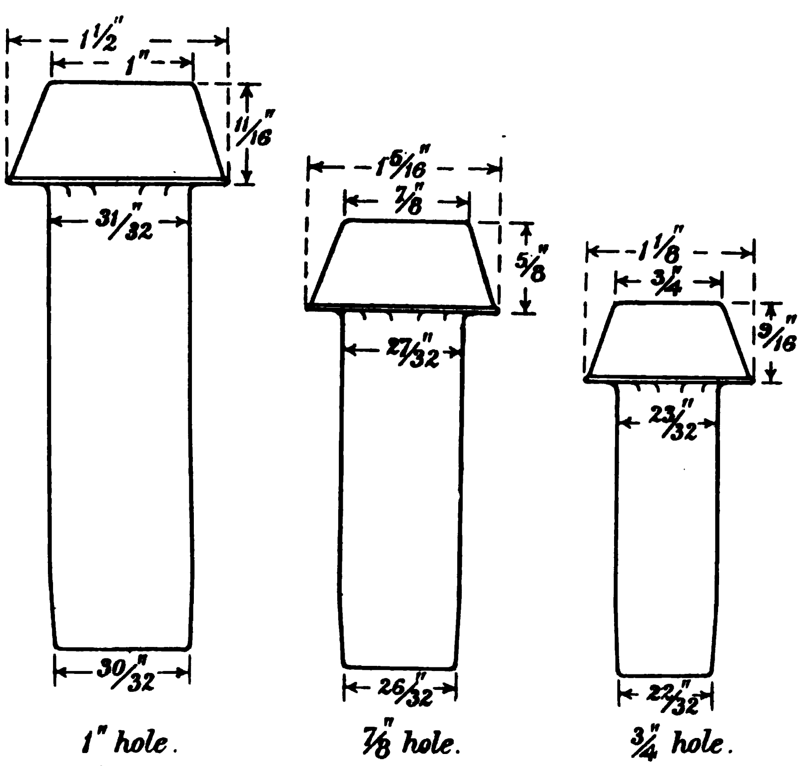
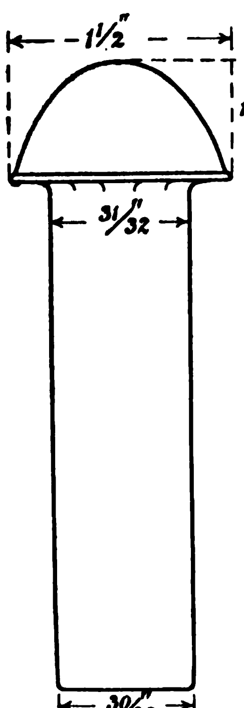


FIG. 24. FIG. 25. FIG. 26.

purposes, and any material error in position of the punched hole is not modified in the process of drilling.

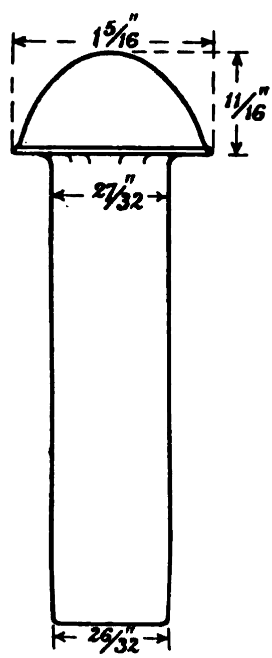
Notwithstanding, as above stated, excellent work is produced by this method, and the accuracy of the holes when assembled together can be made to fulfil all requirements of first-class work, though not equal to that which would result from the process of drilling through all thicknesses at once.

Certain roughnesses left on the surface of the steel plate or



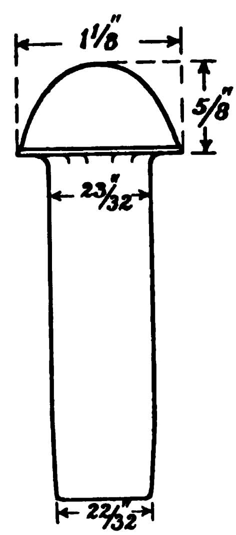
1" hole.

Fig. 27.



⅞" hole.

Fig. 28.



¾" hole.

Fig. 29.

bar at the edges of the holes as the tool enters or emerges from the hole should be scraped off before the meeting surfaces are placed together for riveting, as they tend to prevent close contact.

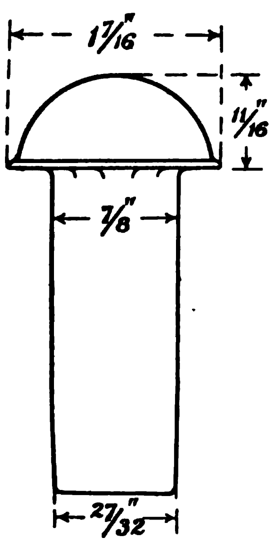
Details of the mechanical tests of mild steel for rivets are given in Table No. 14, p. 41, and the chemical analysis of a sample for the same purpose will be found on p. 52.

A comparison of the rivets manufactured by various makers

and commonly used in constructional steelwork will show certain variations in the shape and dimensions of the heads.

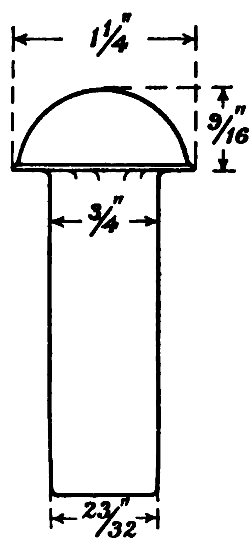
Figs. 24 to 32 have been drawn and measured from actual specimens as manufactured and used by well-known firms in this country. As a rule, mild steel rivets have heads and points somewhat heavier than those employed for wrought-iron rivets.

Figs. 24, 25, 26, show pan-headed rivets, and Figs. 27, 28, 29, show one type of cup-headed rivets, while Figs. 30, 31, 32, show cup-headed rivets of somewhat different shape. For the



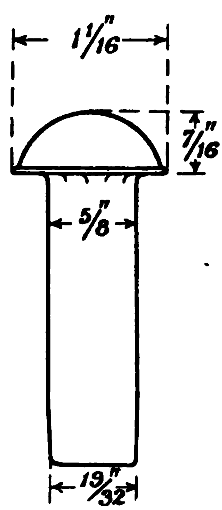
7/8" rivet.

FIG. 30.



3/4" rivet.

FIG. 31.



5/8" rivet.

FIG. 32.

combination of punched and rimered holes above described there will be a difference of about $\frac{1}{32}$ inch between the diameters of the hole and the rivet to allow for entry. The student will consequently appreciate the distinction between a rivet figured for, say, 1 inch hole, and a rivet figured as 1 inch diameter, and as by the terms of most specifications the rivet is to fill the hole, a rivet, say, of $\frac{31}{32}$ diameter may for purposes of calculation be reckoned as 1 inch diameter when completed.

In the estimation of weights of structural steelwork the weights of the heads and points of rivets (except where countersunk) must

be allowed for. The actual percentage will vary slightly in different classes of work, being greatest in those cases where the riveting is heavy relatively to the thicknesses of plates connected and the pitch close. About 4½ per cent. will, as a rule, be found sufficient for heavy girderwork, but a more reliable estimate in individual cases is arrived at by counting the rivets where practicable and allowing the values given in the following table, in which the point or snap of the rivet is assumed to be of the same weight as the type of cup-head shown in Figs. 27, 28, 29.

TABLE No. 30.

THE WEIGHT OF HEADS AND POINTS OF MILD STEEL RIVETS.

Diameter of rivet	½″	⅝″	¾″	⅞″	1″
Weight of head and point of rivet in pounds per hundred rivets	9	13	24	35	50

Tables of the weights of mild steel bolts and nuts are given at the close of this chapter.

While the older-fashioned methods of hand-riveting are still employed in those situations or under those conditions which require them, yet the great bulk of riveting is now carried out by machine-work, the power employed being usually either hydraulic or pneumatic. In the former process a steady pressure is applied to the heated rivet, which, if allowed to remain on long enough, produces a thorough filling up of the hole in a manner which cannot be surpassed. In the pneumatic or compressed air method the process may either be one of steady pressure, or of a succession of rapid blows produced by the tool known as the pneumatic hand, hammer, which, albeit somewhat noisy, has proved its efficiency in this direction, while similar processes are applied to caulking, drilling, and other mechanical work. A pneumatic holder-up is also used in connection with the hand-hammer, but is often replaced by the older hand method where convenience requires.

Some difference of opinion exists as to which of the methods, hydraulic or pneumatic, as applied to hand-hammers, produces the soundest work in closing up the rivet. There is no doubt that good work can be produced by either mode, and the formation of

the snap-head by the hand-hammer can be completed with great neatness and finish.

The pneumatic hand-hammer also finds a place in certain processes as much associated with architecture as engineering, being used in the dressing and carving of stonework.

Girderwork as applied to Bridge Construction.—It is obviously impossible in a collection of notes such as the present to deal even in the most elementary manner with the details of bridge construction in steel. The subject is one of immense extent, and the details of even one such structure of the first class and of very large span would suffice to fill a volume of itself.

The example here chosen is selected merely as typical of the application of plate girderwork to a comparatively small structure of short spans, but as the work includes some other useful details of various kinds, it will be further discussed.

The structure in question forms a viaduct connecting certain outlying jetties and wharves with the mainland.

The roadway is therefore designed to carry a mixed traffic of foot passengers, railway lines, and ordinary road vehicles, together with certain provision made for pipe-work, such as hydraulic mains, etc. This combination required an arrangement of road-bed or bridge-floor adapted to meet the requirements of the conditions above described.

Fig. 33 gives an elevation of one span of the viaduct, which is formed of a pair of heavy plate girders supported on cast-iron cylinders placed 60 feet apart longitudinally, and 25 feet 9 inches centre to centre transversely, as shown in Fig. 34, which is a cross-section of the viaduct at the centre of one of the spans, the clear width of roadway between main girders being 24 feet.

The cast-iron cylinders were constructed in lengths of 6 feet in height as a rule, certain special or make-up lengths being supplied to reach the prescribed finished level at the girder-beds in accordance with the slightly varying depths to which the cylinders were sunk, determined by the nature of the strata reached, and by the amount of settlement of the cylinder under the prescribed test-load.

These lengths of cylinder were each cast in one complete ring without vertical joint.

This method of construction can be easily carried out up to about 10 feet or thereabouts in diameter. For larger diameters it is usual to cast them in segments with vertical

joints, a system which offers some advantages for shipment abroad.

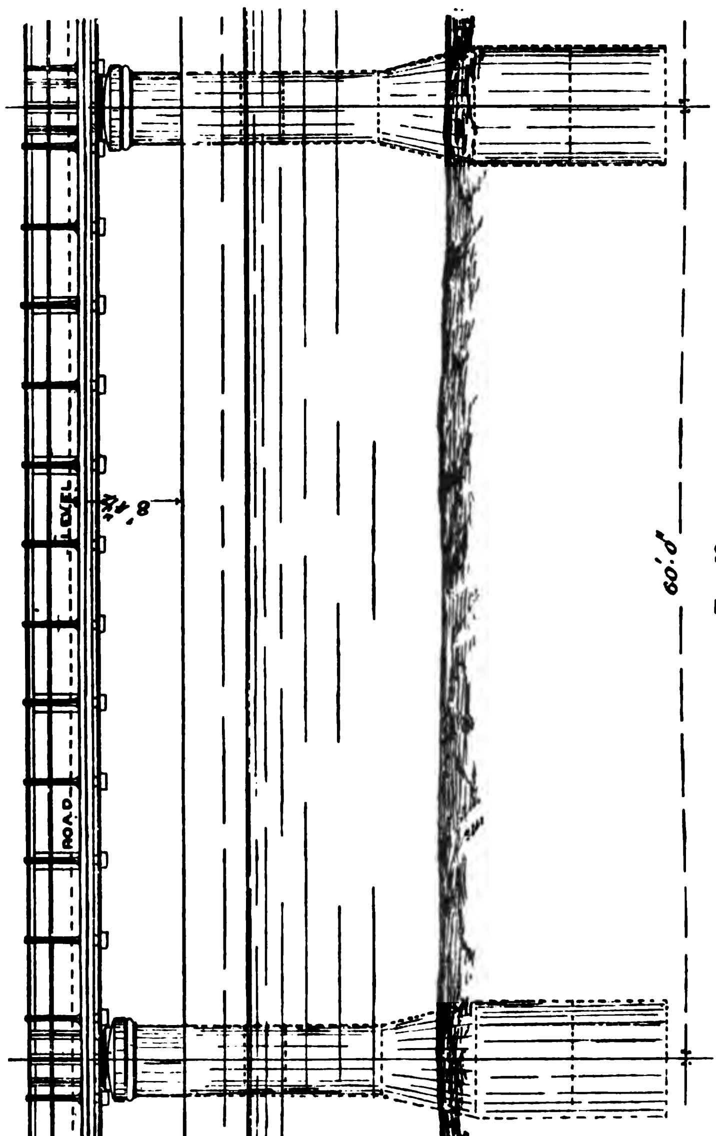


Fig. 33.
Scale 1 inch = 12 feet.

The bottom lengths are 7 feet 6 inches in diameter. Above these

is the taper length shown in Figs. 33 and 34, leading to a reduction in diameter for the uppermost lengths which maintained a uniform

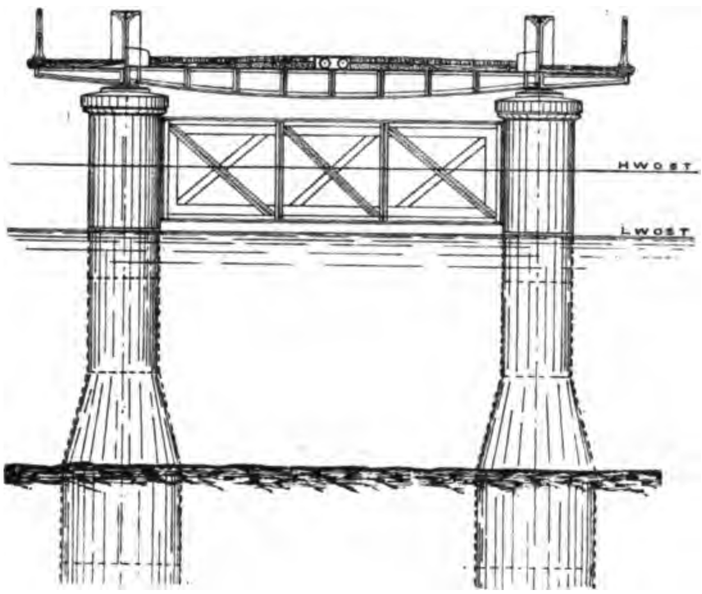
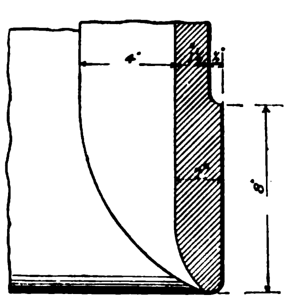


FIG. 34.
Scale 1 inch = 12 feet.

diameter of 4 feet 6 inches up to the level of the capping which forms the capital of the column. The lowermost of the bottom lengths is furnished with a cutting edge, as shown in Fig. 35, for convenience in penetrating the strata through which the cylinder has to pass, being sunk by the combined processes of undercutting at the cutting edge, and forcing down by dead weight applied at the top, the interior of the cylinder being kept dry by the use of the compressed air system, an air-lock being used for passage into and out of the cylinder.



DETAIL OF CUTTING EDGE.

FIG. 35.
Scale 1½ inch = 1 foot.

The enlarged diameter at the bottom of the cylinder affords facilities for the necessary excavation.

The lengths of cylinders were stiffened by internal vertical ribs as shown in Fig. 36, which also shows the nature of the horizontal

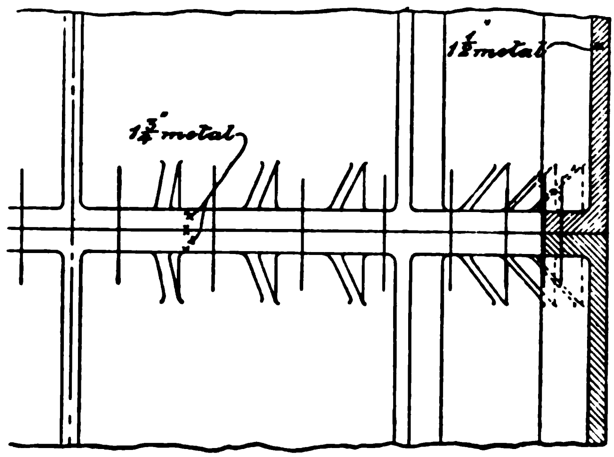


FIG. 36.
Scale ¾ inch = 1 foot.

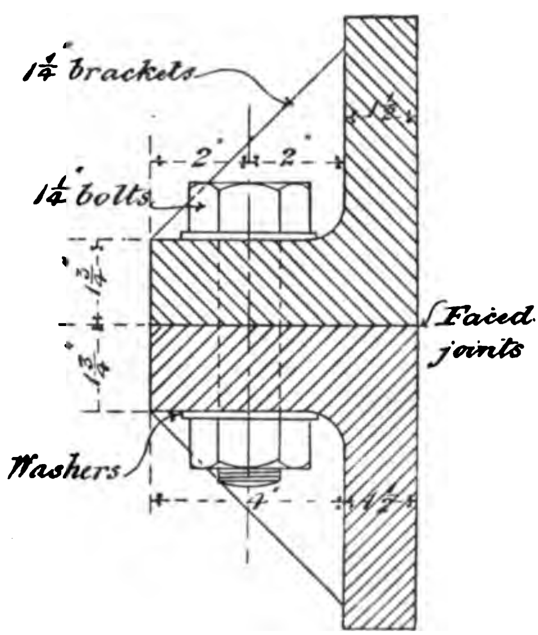


FIG. 37.
Scale 3 inches = 1 foot.

joint and bolted connection between the bottom lengths, the section of the joint being further shown in detail in Fig. 37.

It will be observed that between every bolt a triangular stiffening bracket is cast connecting the flange with the metal skin of the cylinder.

The flanges are machined for their entire width, while all the bolt-holes are drilled, ensuring sound work and the precise duplication of the joints.

The water-tightness of these joints is secured by the use of canvas and red lead, or by an indiarubber ring about $\frac{1}{4}$ inch

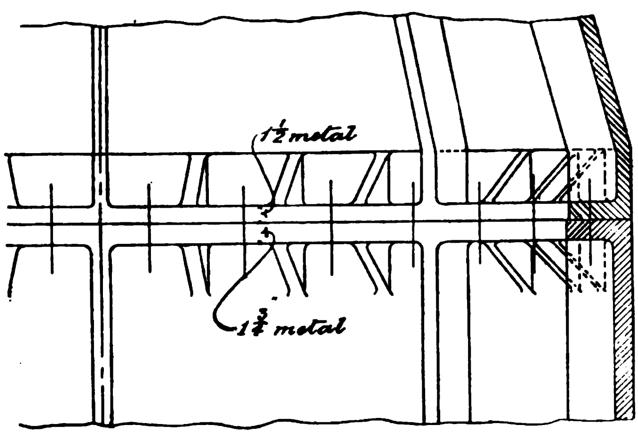


FIG. 38.
Scale $\frac{3}{8}$ inch = 1 foot.

diameter placed between the machined faces and squeezed out by the bolting up of the joint.

It has occasionally happened that either from the existence of initial cooling stresses in the casting, or from certain inequalities of stress arising from the forcing down of the cylinder through hard and difficult strata, the bottom length of cylinders such as those now under consideration have cracked more or less seriously during the process of sinking, and this has led some designers to adopt a riveted form of construction for the lowermost length.

The horizontal joint between the lower lengths and the taper or conical length next above them is shown in Fig. 38.

The joints of the upper lengths of reduced diameter are similar to that shown in Fig. 37.

The upper portion of the cylinder at the level of the capping and girder bedstones is shown partly in elevation and partly in section

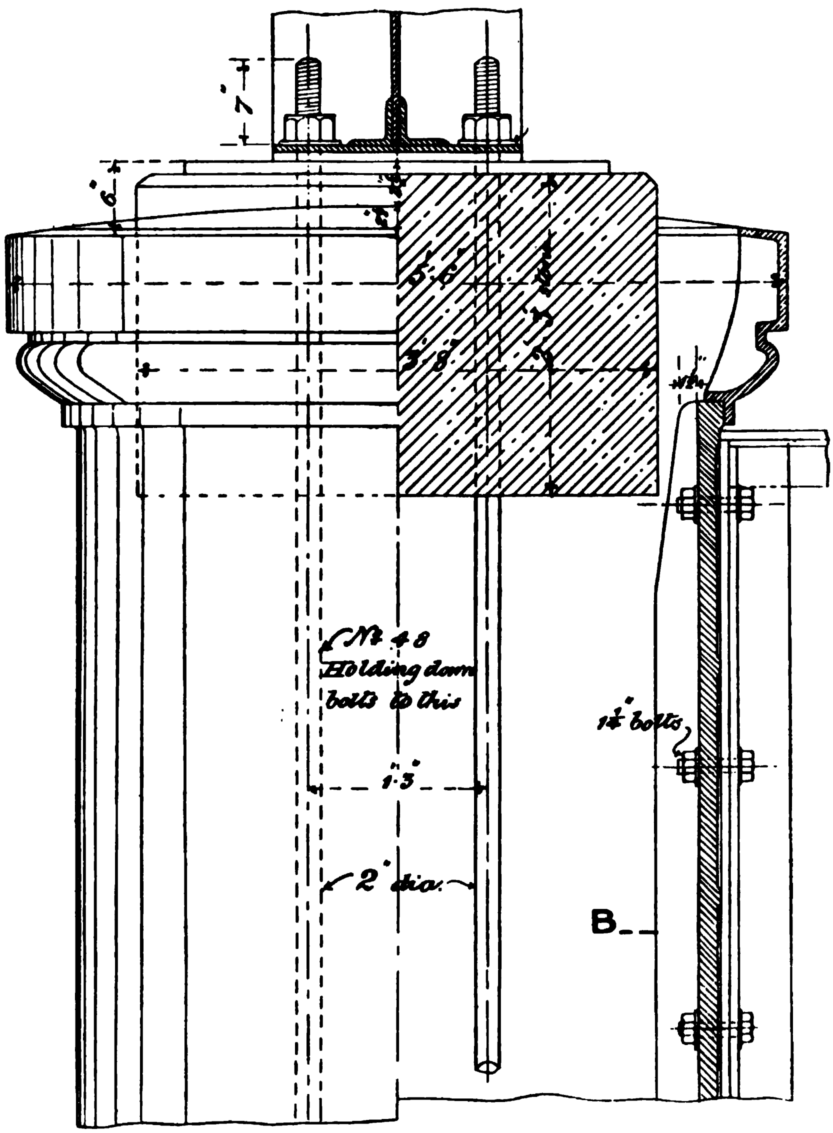


FIG. 39.
Scale ¾ inch = 1 foot.

in Fig. 39. The moulded cap or capital is cast separately from the

cylinder length, is of ¾-inch metal, cast in a convenient number of segments, and is bolted to the top length of cylinder in the manner shown in Fig. 41, which shows a detailed section of the moulding, while Fig. 40 shows the internal elevation at a joint of the segments.

Within the capping, a hard stone girder-bed of the dimensions shown in Fig. 39 is inserted, resting upon the concrete with which the cylinders are filled, to receive the ends of the 60-feet main girders.

The general levels of the work in the vicinity of the viaduct

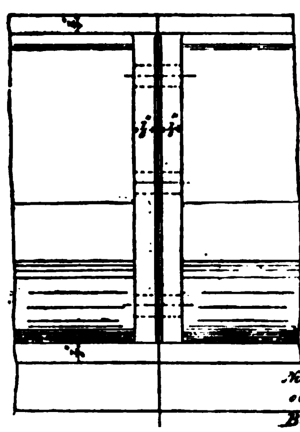


FIG. 40.
Scale 1¼ inch = 1 foot.

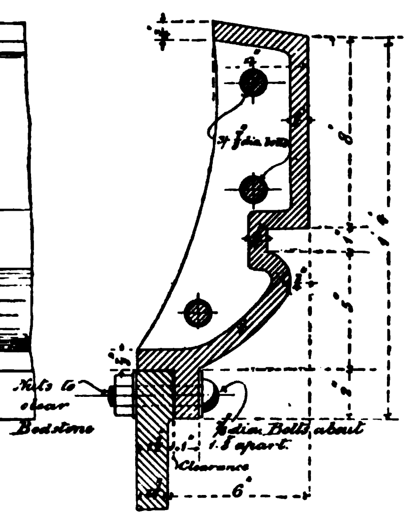


FIG. 41.
Scale 1¼ inch = 1 foot.

did not permit of any greater headway above high-water mark and the underside of the main or cross-girders than that shown in Fig. 34.

In such cases, not infrequent in jetty work, it becomes desirable to counteract a possible uplifting force from beneath caused by the displacement of floating craft, such as barges, which may by mischance have been caught underneath the girders on a rising tide, and tending to displace the girderwork above them.

This is effected by the holding-down bolts 2 inches in diameter, shown in Fig. 39, passing through the bottom flange of the main girders and the bedstones, and carried down a sufficient distance

into the concrete filling of the cylinder, and having at their lower ends the cast-iron ribbed washer-plates shown in Fig. 42 and Fig. 43.

These bolts are carefully fixed in position by templets, and with their cast-iron washers embedded in the concrete as the filling progresses, the girder-beds being slipped over them. To allow for possible errors in the levels of setting, the screwed ends are kept well above the nuts as shown. This is a wise precaution wherever foundation bolts are liable to displacement by sinking during the progress of the work, and allows a margin of error in

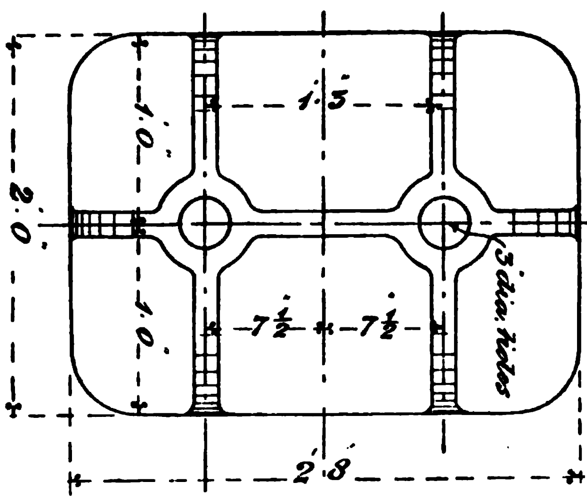


Fig. 42.
Scale 1 inch = 1 foot.

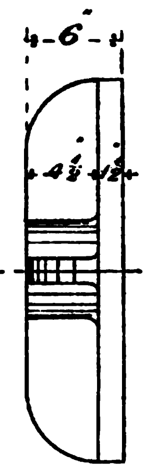


Fig. 43.

construction. The excess of length can, if thought desirable, be cut off after the work is completed.

The seating of the ends of the main girders with their holding-down bolts is also shown in elevation in Fig. 44, and in sectional plan in Fig. 45.

The entire cylinder was filled with Portland cement concrete, after the sinking was completed and the bottom ascertained to be satisfactory. The cylinder was then loaded at the top with a dead weight of pig-iron.

The transverse resistance of cylinders, in such a situation as

the present, when exposed to the shock of a bump from floating craft or other force tending to shift the cylinders laterally, must

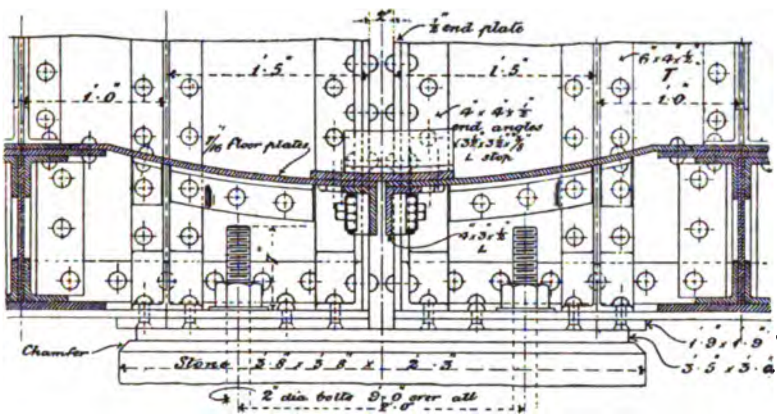


Fig. 44.
Scale ¾ inch = 1 foot.

always be carefully considered, and is usually met by the adoption of a system of bracing, which will vary in detail in accordance with the circumstances of the case.

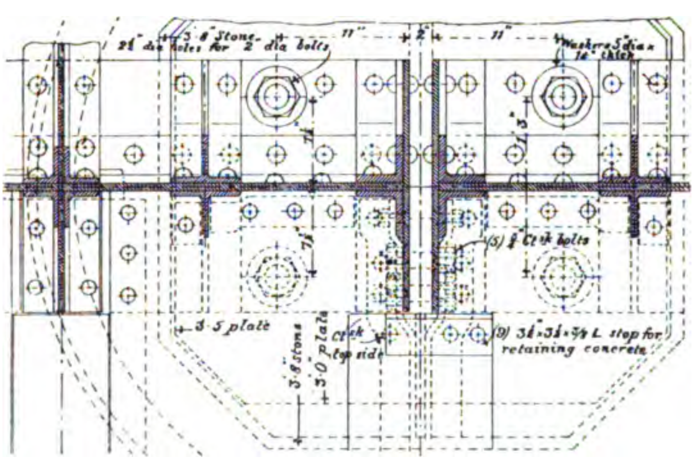


Fig. 45.
Scale ¾ inch = 1 foot.

The efficiency of such a system of bracing will largely depend upon its depth, but in many situations it may be advisable to

avoid underwater connections, which can only be made by diver's work, and are not easily accessible for examination or repair. Such a course is frequently not open to the designer of marine work, but in the present instance the method of bracing shown in Fig. 34 was adopted as affording a practically efficient depth of brace, while the connections with the cylinders could be made without much difficulty without diver's work.

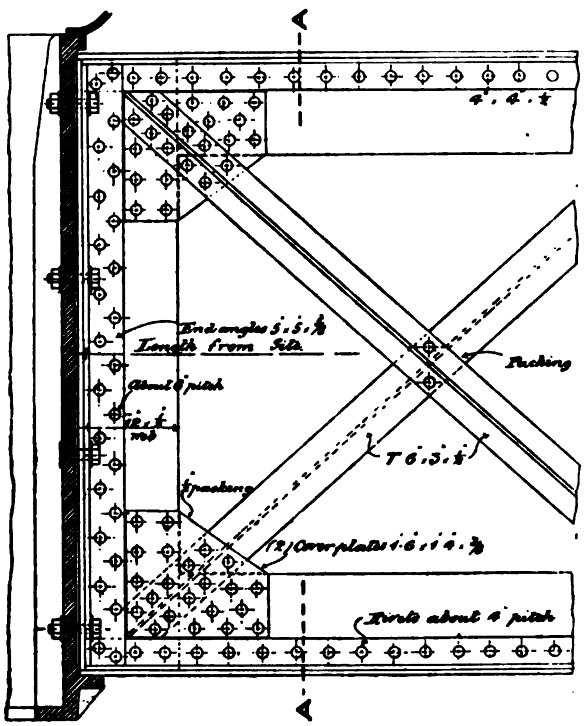


Fig. 46.
Scale ¼ inch = 1 foot.

The bracing consisted of a deep lattice girder as shown in Fig. 34. The exact length of this girder between the cylinders was obtained after the latter were sunk, to ensure accuracy of fitting. Any inaccuracy in the distance apart of the centres of the cylinders could be adjusted within limits, as far as the main and cross-girders were concerned, by an adjustment of the girder-beds and bolts within the cylinders themselves.

The detail of connection of the lattice girder with the cylinder is shown in elevation in Fig. 46, while Fig. 47 shows the detail of the bolting up in section on the line B in Fig. 39.

The centre bay of the braced girder is shown in elevation in Fig. 48, and vertical cross-sections of the girder are shown at the end and at the centre respectively in Figs. 49 and 50.

The main girders over the 60-feet openings were of the single-web type, designed to carry the heaviest loads which could arise either from ordinary road traffic, wheeled and passenger, or from

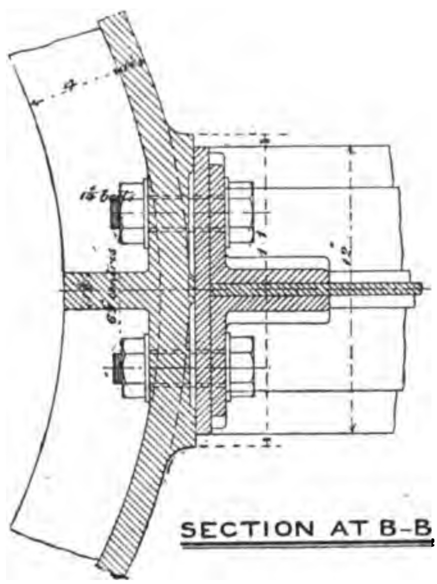


FIG. 47.
Scale 1¼ inch = 1 foot.

the heaviest rolling load on the rails which could be anticipated to arise.

The section of these girders at the centre is shown in Fig. 51, which also shows the attachment of the cross-girders supporting the roadway, and of the plate girder cantilevers supporting a footway for passengers on both sides of the viaduct.

The mode of giving rigidity to the fixed end of the cantilever is indicated in the figure, this end being securely clamped between the bottom flange of the main girder and the plate stiffener which occurs at every cross-girder and cantilever, 5 feet apart, and

securely riveted to both, as well as the web—a form of construction sufficiently rigid for the maximum load which can come upon it in this case.

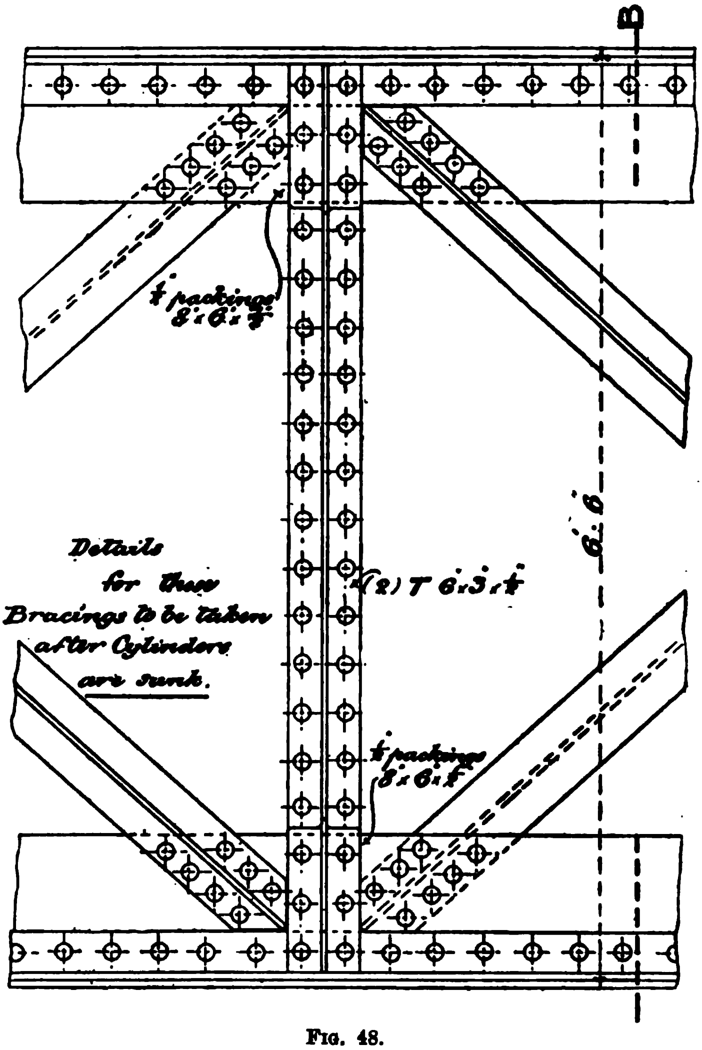


Fig. 48.
Scale ⅜ inch = 1 foot.

A timber bolster is bolted on the top flange of the cantilever, to which is spiked the 2½-inch timber flooring forming the footway.

The outer ends of the cantilevers are connected with a continuous angle-steel carrying a moulded timber fascia, upon and through which are bolted the cast-iron handrail standards with gas-tube handrails, all as shown in the figure.

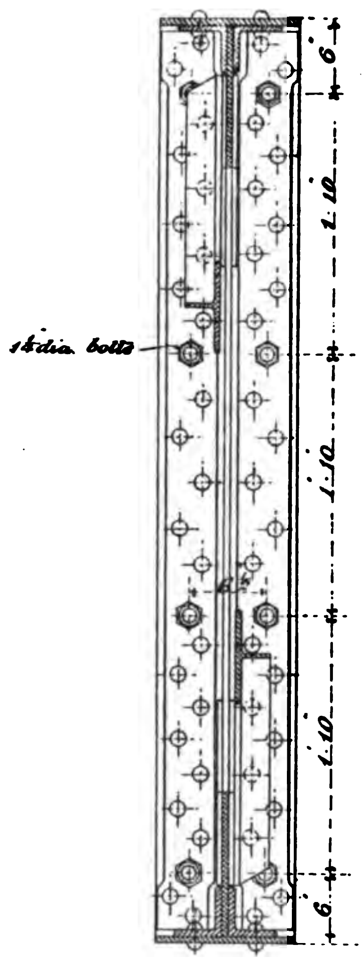


Fig. 49.
Scale ¾ inch = 1 foot.

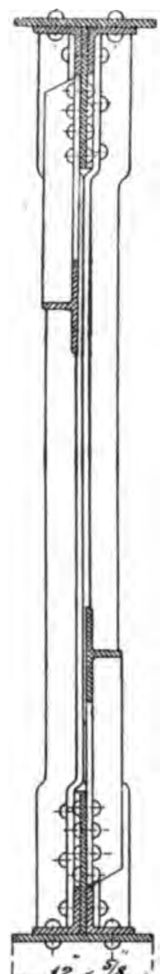


Fig. 50.
Scale ¾ inch = 1 foot.

The floor of the viaduct is carried by fish-bellied cross-girders, as shown in Fig. 34, spaced 5 feet apart, as in Fig. 33. The

section of these girders is shown in Figs. 44, 52, and 54. The connection of the cross-girder with the main girder is shown in

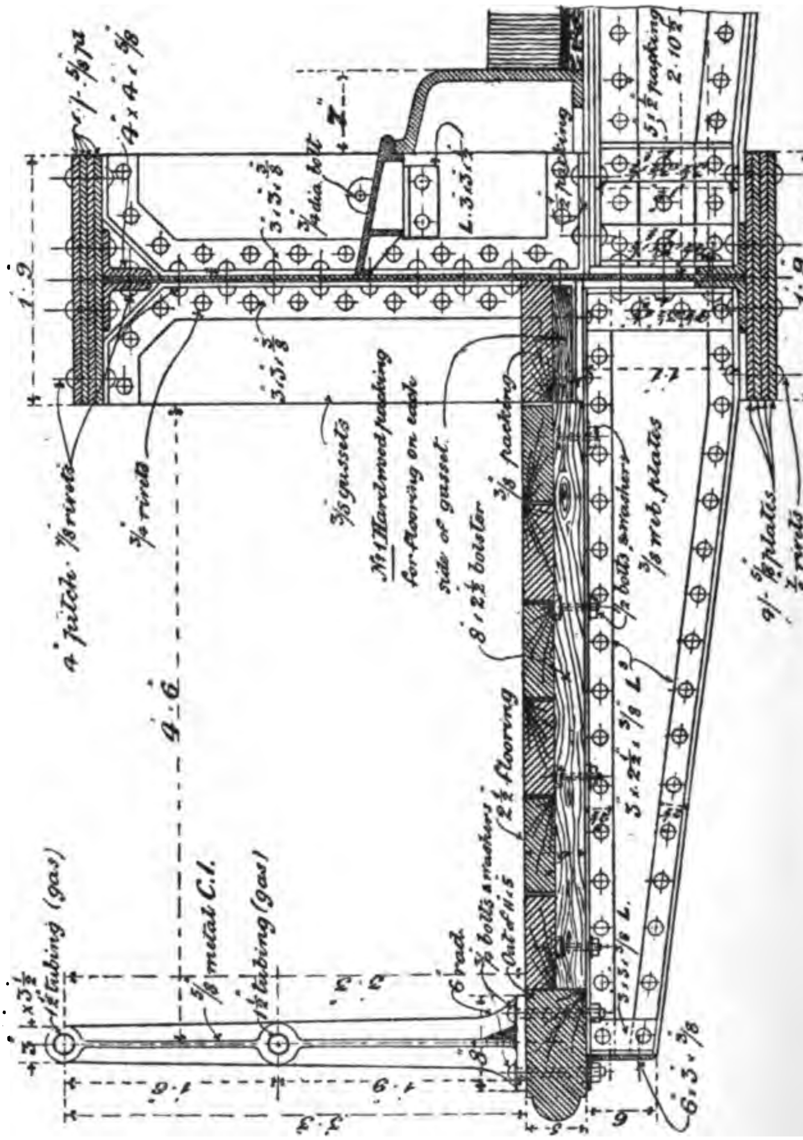


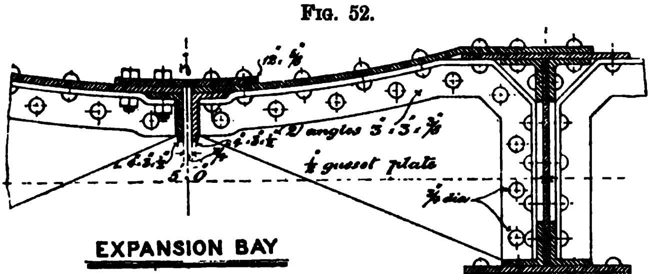
FIG. 51.
Scale $\frac{3}{4}$ inch = 1 foot.

Fig. 51, the plate stiffener of the main girder being stopped on

the upper flange of the cross-girder and riveted to it as shown. The practical continuity of these plate stiffeners, as between the upper and lower flanges of the main girder, is thus secured on both sides, and the entire combination attains the necessary rigidity. In larger and more important structures the cross-girders and cantilevers for the footway would probably be in one, and supported below (or in some cases above) the bottom flange or tension member of the main girder.

Upon these cross-girders are laid bent floor-plates riveted to the upper flange of the cross-girder as shown, the plates being concave in this case, and not convex as in the older-fashioned buckled plate. The joints of the floor-plates longitudinally are

FIG. 52.



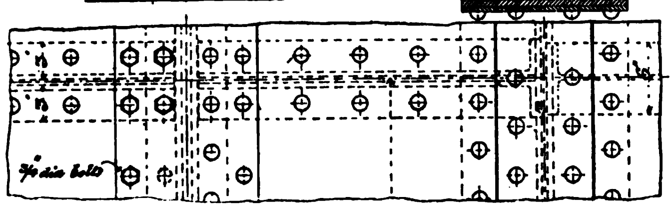


FIG. 53.
Scale $\frac{3}{4}$ inch = 1 foot.

covered by tee-steels, which are continuations of the web stiffeners of the cross-girders as shown.

The junction of the floor-plates of two adjacent spans, where the main girders meet over the piers or cylinders, is shown in Fig. 44.

This may be considered as a "fixed" end for expansion purposes, while the "free" end, for expansion due to change of temperature, is shown in section and plan in Figs. 52, 53, the space between the plates being covered by a $12'' \times \frac{5}{8}''$ strip, riveted on one side, and bolted on the other, in slotted holes so as to allow a certain amount of freedom of movement. But the arrangements for

expansion and contraction in a structure of these moderate dimensions are not of the same importance as in bridge-work of a larger class.

The floor-plates are flushed up to a level surface by concrete filling, as shown in Fig. 54, and upon the surface thus formed wood-block paving is laid in the usual manner, upon a layer of asphalte.

The rails are of steel of flat-footed section, weighing 75 lbs. per yard, and are laid on cross-sleepers embedded in the concrete, the space being laid between them in wood blocks as shown, the nature of the traffic generally not allowing of the rails standing up above the general level of the roadway. Surface water

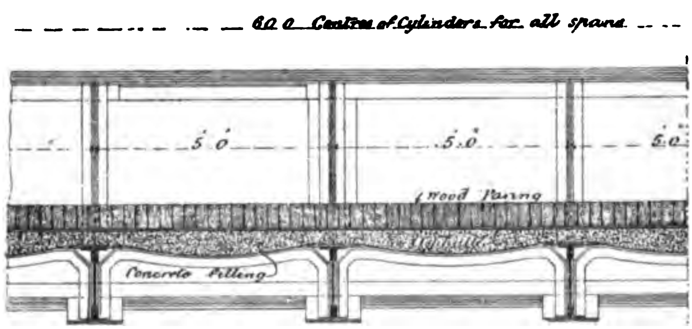


Fig. 54.
Scale ¼ inch = 1 foot.

is disposed of by drainage outlets at intervals, discharging beneath the bridge into the sea below.

A trench is laid in the centre of the roadway with cast-iron cover-plates, to receive pipe-work of comparatively small diameter, such as hydraulic or small gas or water pipes, or electric mains. Any other pipes of larger dimension, such as sewage or large water-pipes, would require to be specially provided for, and would probably be slung under the cross-girders, or, as in some cases, carried alongside the main girder next the footway, and cased in with timber framework and boarding, with occasional doors for examination or repairs.

In cases such as the present, where the roadway is carried between the main girders (which form a parapet), and mixed wheel traffic has to be provided for, it is necessary to protect the plate stiffeners of the main plate girders, or the tension or

compression members of a braced structure, from blows which might be received from the hubs or other projections of a wheeled

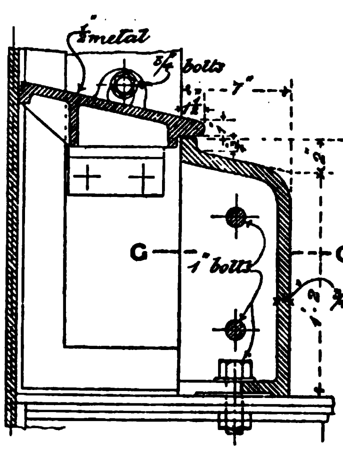


FIG. 55.
Scale 1 inch = 1 foot.

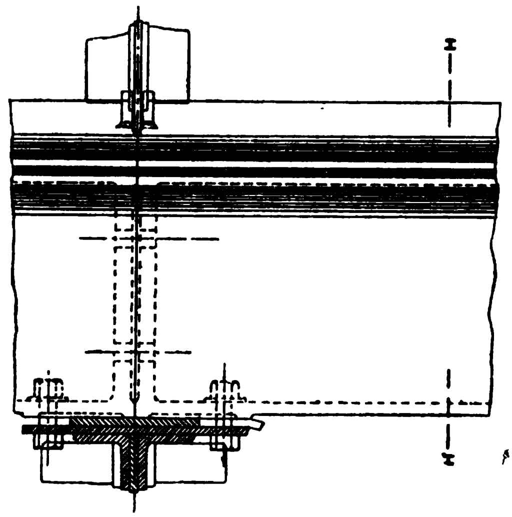
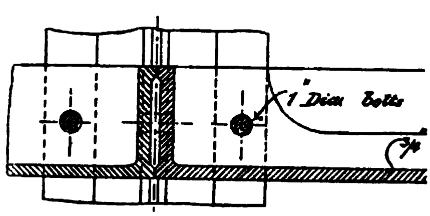


FIG. 56.
Scale 1 inch = 1 foot.

vehicle. To effect this the wheeled traffic is kept off the girderwork by the cast-iron curbs shown in section in Fig. 51, and in further detail in Figs. 55, 56, 57, and 58. Fig. 55 is an enlarged section of the curb, with its capping running back to the plate-web of the main girder, thus covering in the void between the curb and girder. The joints between this cover and the plate-web and stiffeners may be caulked with oakum and pitch to prevent wet from getting in.

FIG. 57.
Scale 1 inch = 1 foot.



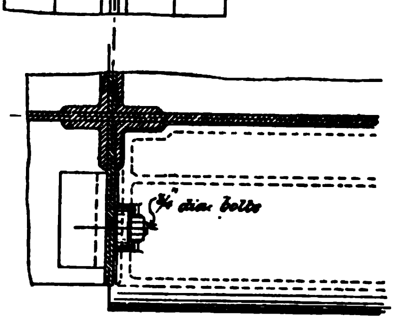


FIG. 58.
Scale 1 inch = 1 foot.

The projection of the curb beyond the face of the stiffeners is in this case 7 inches. A recent metropolitan example under somewhat similar conditions gives 12 to 13 inches for a similar dimension.

The curb may also be constructed of timber, with an angle-iron at the edge to take the rub of the wheels.

Fig. 55 is a section on the line H,H, in Fig. 56, and Fig. 57 is a sectional plan on the line GG in Fig. 55, while Fig. 58 is a part plan showing connection with the plate stiffener, and Fig. 56 is a part elevation showing the joint in the curb at every stiffener, and over a cross-girder.

Girderwork for Machine or Boiler Shops, Steel Foundries, etc.: Traveller Girders.—An important application of steel girders, whether of the simple rolled-joist, single-web, double-web, or box type, is found in the longitudinal girders forming the roadway or railroad for overhead travelling cranes.

The loads lifted by these cranes are of all degrees of magnitude, varying from a few hundredweights up to the loads of 100 tons or even more, found in steel foundries, gun-making establishments, boiler or machine shops, and the like.

The height to which such loads have to be lifted are as variable as their weights, and in the heaviest cranes the level of the crane-rail may be as much as 40 to 50 feet above floor-line. In such cases the details of columns supporting such crane-loads become worthy of consideration, and the entire structure requires careful design in view of the important results which would arise from failure either in the columns or roadway.

The loads placed by crane-makers upon the wheels of such structures as overhead travelling cranes, titans for marine construction, heavy locomotive wharf cranes, and the like, considerably exceed anything found in ordinary railway practice or locomotive construction.

This is probably explainable by the fact that the speeds of such structures in locomotion are very slow as compared with the ordinary forms of rolling stock.

Thus, to take an illustration, the total weight of an overhead travelling crane of about 60-feet span driven by shafting, and capable of lifting 35 tons, was found to be 42 tons exclusive of the load lifted. This crane is supported by a cradle at each end fitted with two wheels, making four wheels in all. Thus the load per wheel of the crane only with the crab in the centre was 10·5 tons;

with the load lifted in the centre, the load per wheel amounts to 19·25 tons; but with the load lifted at, say, 6 feet from one crane-road, the maximum wheel load at the heavy end runs up to approximately 30 tons per wheel, the exact amount depending on the precise ratio between the fixed and moving parts of the crane, or as between the transverse girders and other equally distributed weights and the crab.

In some titans for breakwater construction wheel loads of about 40 tons per wheel are not uncommon in certain positions of the load.

Such loads as these demand considerable attention to the section of rail used and mode of fastening, as the tendency under such moving loads as these is to curl up the rail even when of fairly heavy section, and tear it from its fastenings—if the foundation be at all of a yielding nature. When, however, a sufficiently good and rigid connection has been made, there is no difficulty in dealing with wheel loads of the amount named.

The structure of the wheel itself is somewhat outside the limits of this work, but in general it may be stated that such wheels are usually made of cast iron in a solid form, or with a few simple round holes in the web of the wheel, with Bessemer or Siemens steel tyres of massive section shrunk on.

In many cases, such as wharf cranes, titans, or goliahs, a wheel is used having one central flange, and two treads, requiring a double rail, or two rails placed side by side to run on, which forms a very satisfactory arrangement, but in the ordinary cases of overhead travellers, the rail is single, and the wheel may be either single or double flanged, usually the latter.

The following table, No. 31, gives the approximate total weight of overhead travelling cranes, exclusive of the load lifted, for spans varying from 30 to 60 feet, and for loads of from 5 to 100 tons.

These weights are approximate only, and may be taken to cover the various types of shaft-driven, rope-driven, hand, or electric cranes. The values given are only sufficiently accurate for preliminary calculations, and in all important cases the actual probable weight of the crane should be obtained from the makers, together with other information hereafter referred to.

In connection with this subject it may be mentioned that a recent example of a steam crab to lift a test load of 50 tons, as applied to a goliah, was found to weigh 32 tons 3 cwts. 1 qr. 19 lbs., including boiler, house over crab (for outdoor work), lifting beam

and rods, eye-bolts, snatch-block, and wire-rope, but exclusive of coal and water. The test load was lifted at a speed of 10·3 feet per minute, and was traversed at 33 feet per minute, the whole goliah with its load weighing in all about 137 tons, being travelled at 120 feet per minute. Pressure of steam in boiler, 80 lbs.

TABLE No. 31.

Span in feet.	Power of crane, or load lifted in tons.											
	5	10	15	20	25	30	35	40	50	60	80	100
30	12	16	19	22								
35	13	17	20	24								
40	14	18	22	27	31	34	37	40	46			
45	15	19	23	28	32	35	38	42	48			
50	16	20	24	29	33	36	40	43	50	58		
55	17	21	25	30	34	38	42	45	52	60	74	
60	18	22	26	31	35	40	44	47	55	62	76	90
	Total weight of crane, exclusive of load lifted, in tons.											

Before the actual maximum wheel loads can be arrived at, and the determination of bending moments due to the rolling load made, it is necessary to subdivide the total weight of crane into the fixed and moving portions relative to the two crane-roads. Thus the fixed portion will consist of the pair of supporting girders spanning the distance or gauge between longitudinal roadways, together with such other portions of the gear, shafting, etc., which may be evenly divided between the two end cradles. The moving portion will consist of the crab for lifting the load, which may take any position between the two crane-roads, the lateral amount of travel being limited by the minimum distance which the crab with its load can assume from the rail, a distance which may be determined either by the details of the crab itself or the dimensions of the load to be lifted.

When these proportions of load are known, the relative reactions at the cradles or the wheel loads can be easily ascertained, and the bending moments on the girders due to the rolling loads

can be determined, the most effective means being by graphic analysis.

For approximate and preliminary calculations, and in the absence of more precise information to be obtained from the manufacturer, the total weight of the crane given in the table may be divided evenly into the fixed and moving portions. For example, the weight of a 35-ton traveller of 55-feet span being 42 tons, 21 tons may be considered as evenly divided between the two cradles, and 21 tons may be taken as the weight of the moving crab with its gearing, which may occupy any position laterally between the crane-roads, giving rise to proportionate reactions or wheel loads, as the position may determine, in accordance with the principles of the lever.

In some cases two travelling cranes of equal power may be temporarily coupled together to lift a load equal to twice the load lifted by one traveller alone. In this case the disposition and spacing, and the total wheel-base of the wheel loads, will be determined by the dimensions of the end cradles, and that position of the total load must be ascertained which gives rise to the maximum reaction in the supporting column, as well as the maximum bending moments in the girder.

The crane manufacturer in the design of a traveller of this class will always require certain important minimum dimensions or clearances to be maintained in the structure in which the crane is to be employed.

One of these is the distance to be maintained between the centre of the rail and the face of the wall, pier, buttress, column, or other projection past which the end cradles have to travel. This dimension is required by the details of the end cradle, and the bearings of the axles of the cradle wheels.

In small travellers of short span and light load the clearance required is about 6½ inches; a clearance of 9 inches will suffice for cranes of about 35 tons lifting power, while 11 to 12 inches will cover most ordinary cases of heavier cranes. This dimension will be found to exercise a considerable influence over the design of columns to support traveller roadways, as will be seen by the examples referred to in Chapter IV.

The other dimension referred to is the headway required, usually measured from the rail level, over the crane, to the lowest fixed portion of the roof or floor above; such, for example, as the distance to be maintained between the level of rails

and the underside of the tie-rod or tension member of a roof principal.

A distinct understanding on this point should always be maintained with the crane manufacturer, who should be asked to state his requirements, while it is judicious at the same time to allow some small margin over the precise figure asked for. In most cases a headway of not less than 7 feet 6 inches will be required for cranes of, say, 30 tons power, the precise amount being regulated by the details of the crab, the diameter and height above rail of the great wheel and main barrel, or possibly, in the case of a steam crane, by the dimensions of the boiler used.

In cranes of small power worked by hand, arrangements are sometimes made by which the traveller can be arranged beneath the roof or floor above with but a few inches of clearance.

The preservation of the truth of the gauge between the rails of a traveller road, from end to end of the distance to be traversed, must at all times claim the attention of the designer of the structure, especially when the spans of the traveller are large, and the supporting columns are lofty.

This is frequently attained in the design of the superstructure, as, for example, in cases where the tension member of the roof principal overhead is designed to act as a tie or a strut, and in such wise rigidly maintain the gauge of the road. An example of this form of construction is given in Figs. 236, 237.

Where the crane road is lofty, and the supporting columns of corresponding height, the *longitudinal* stability of the row of columns and girders must also be considered, and provided for, by bracing, bracketing of the girders, bolting down to foundations, or the like. See Figs. 216, 237.

Where the traveller road is carried by the walls of a building, the stiffness and stability of the masonry or brickwork should be considered in the case of very heavy cranes, and buttresses or piers arranged for as the case may require, the offsetts in the walls being arranged to suit the details of the crane-road and the clearance for cradle above mentioned. In certain types of foundry cranes the stability of the crane is often really dependent upon the stiffness and stability of the enclosing walls of the foundry and its roof framing.

The type and section of girder to be used for a traveller road will be determined by the power and span of the crane to be carried, and the span of the opening to be bridged.

Thus the girder may be of double-webbed or box type, or in some special cases, where the shearing stresses are exceptionally severe, may be of the three-webbed type, while the single-webbed girder may be of any section from a heavy riveted girder to an ordinary rolled joist of light section.

Latticed web or triangulated girders may also be employed subject to great care being taken that the maximum web stresses arising from the rolling load are amply provided for, and also, which is of equal importance, that the upper flange between the apices of the triangulations has sufficient transverse resistance as against the concentrated wheel loads. These considerations generally lead to the adoption of the plate-web type for small spans.

The plate girder may be constructed of uniform depth with parallel flanges, or of the fish-bellied form shown in Fig. 216.

In girders of uniform depth, where the loads and span are considerable, the necessary piling up of the plates towards the centre of the span requires an uniform seating for the rail, which is obtained either by the use of iron packings at intervals, or a continuous strip under the rail, or, as in Fig. 245, by a timber bolster or packing notched to the stepping up of the plates, and trimmed off to a true surface on the top to receive the rail, which may be coach-screwed or spiked to it. The fish-bellied girder, on the other hand, though perhaps rather more troublesome and expensive to construct, possesses the advantage that the sections of the top and bottom flanges being practically uniform from end to end, the upper surface of the top flange plates offers a continuous and even bearing for the rail without the use of packings.

The necessity of a secure fastening for the rail in order to prevent the rolling-up tendency of heavily concentrated wheel loads has already been pointed out.

The section of rail employed is usually either a bridge rail, as in Fig. 245, or a flat-footed rail, as in Figs. 59, 60, 61, 62, 63. The former section has possibly greater stability under the lateral shocks which may occur; the latter section has some advantage in the greater facilities which the bottom flange offers for connection to the supporting girder. In contractors' plant, such as goliahs or titans for temporary work, the rail is very frequently riveted down to the upper flange of the supporting girder, and may in such cases be considered as a portion of the total effective section

of such flange, when no joints occur in the rail at critical points in the bending-moment curve.

Where, however, it is considered desirable to retain the facility of renewing the rail without interference with the structure of the girder, another form of fastening must be adopted in lieu of rivets.

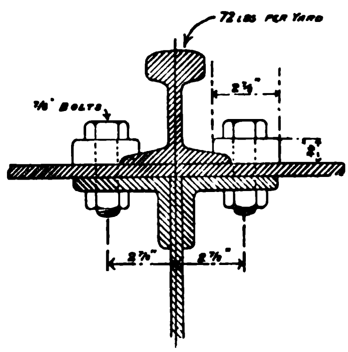


FIG. 59.
Scale 1¼ inch = 1 foot.

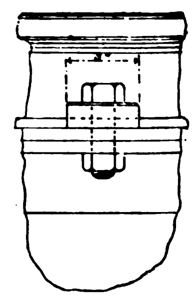


FIG. 60.
Scale 1¼ inch = 1 foot.

A convenient form of such a connection is shown in Figs. 59 and 60, and is repeated under slightly varying conditions in Figs. 61, 62, 63, 64.

It consists of a clip, which may be of cast iron or shaped in mild steel, of a form to suit the exact outline of the rail used, and

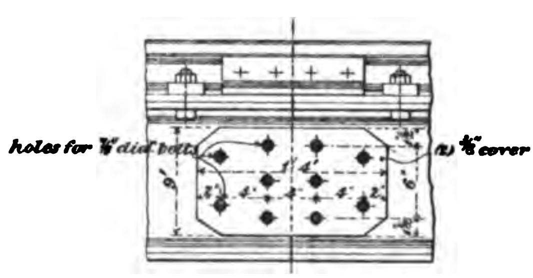


FIG. 61.
Scale ¾ inch = 1 foot.

of length sufficient to enable a bolt to be used which can be properly spaced so as to pass through the flange plate and angles of the single-webbed girder used in this case.

In Fig. 64 the bolt is tee-headed so as to clear the edge of the

flange of the rolled joist. In Fig. 63, where the section of the traveller girder is the rolled joist without any additional flange plates, the bolt is so spaced as to pass through the joist flange, with as much metal outside the hole as can be obtained, while the edge of the rail flange is notched to receive the bolt. Were it

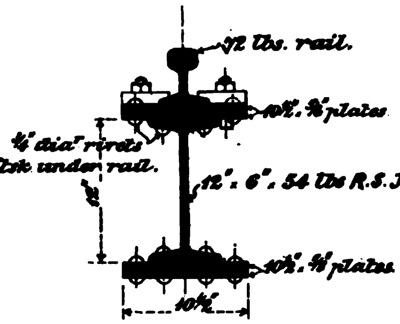


FIG. 62.
Scale ¾ inch = 1 foot.

always possible to obtain a section of rail having just that amount of foot which the designer would prefer for his connection, the design would often be simplified. Unfortunately, from the designer's point of view, it frequently happens in rail sections,

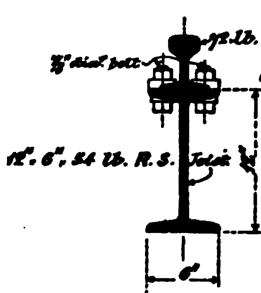


FIG. 63.
Scale ¾ inch = 1 foot.

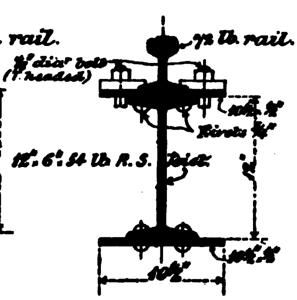


FIG. 64.
Scale ¾ inch = 1 foot.

when applied to work of this kind, that only that section can be adopted which is commercially obtainable at the moment.

In Fig. 245 the bridge section of rail shown is secured to the timber (elm) bearer by coach screws, an iron flat, 9″ × ⅜″, being

placed under the rail to increase the bearing and lessen the indenting stress upon the timber under the rolling load.

Returning to the consideration of the type, sections, and details of the traveller girders themselves, we have in the illustrations appended a few of the methods of construction in frequent use.

An example of the fish-bellied single-webbed plate girder is given in elevation in Fig. 216,, and in section in Figs. 65, 66, Fig. 65 being a section at end, and Fig. 66 a section at centre of

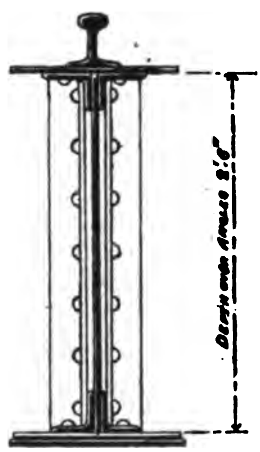


FIG. 65.
Scale ¾ inch = 1 foot.

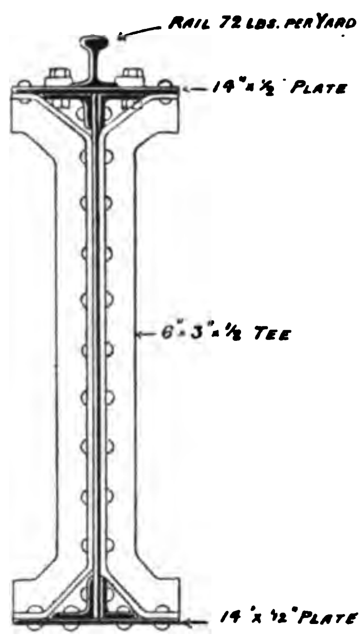


FIG. 66.
Scale ¾ inch = 1 foot.

girder. The section at the centre of a pair of these girders is given in Fig. 67. It will be observed in this example that opportunity is taken in the contiguity of the pair of girders to secure additional lateral stiffness, under accidental shocks from the crane, by means of the pair of struts formed of 3″ × 3″ × ½″ angles bolted on to the plate stiffeners of the main girders.

The bolted connection, combined with sufficient play in the bolt-holes, enables the pair of stiffening bars to act in some degree

as a parallel ruler whenever either of the traveller girders is slightly deflected below the level of its neighbour by a rolling

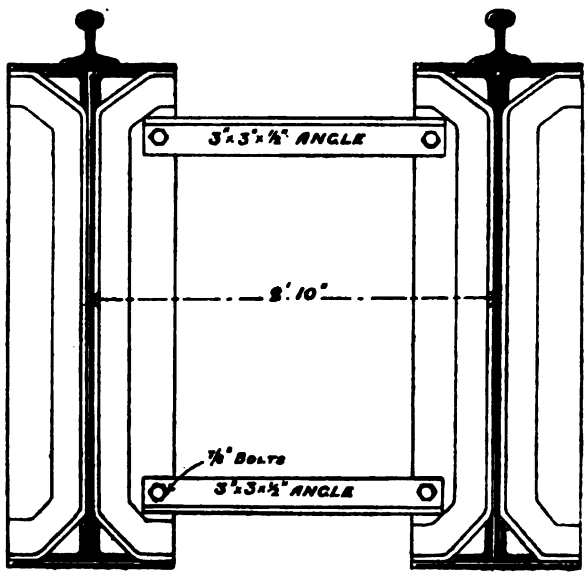


FIG. 67.
Scale ⅜ inch = 1 foot.

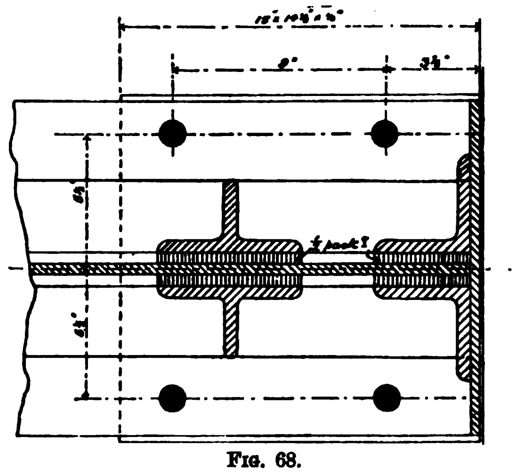


FIG. 68.
Scale 1½ inch = 1 foot.

load. The advantages of the adoption of a fish-bellied form as

regards practical uniformity of section of the top flange, and a consequent simplifying of the rail connections, has already been

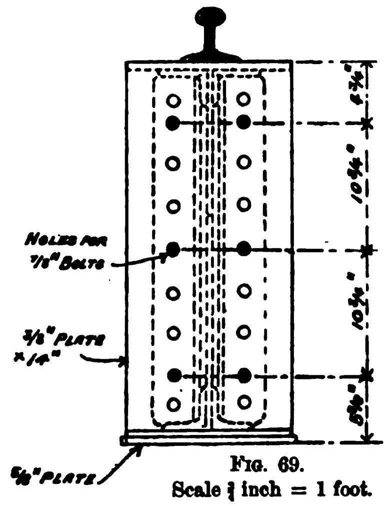


FIG. 69.
Scale ¾ inch = 1 foot.

dwelt upon. The construction is a little more troublesome in manufacture, owing to the necessity of shearing the web plates to

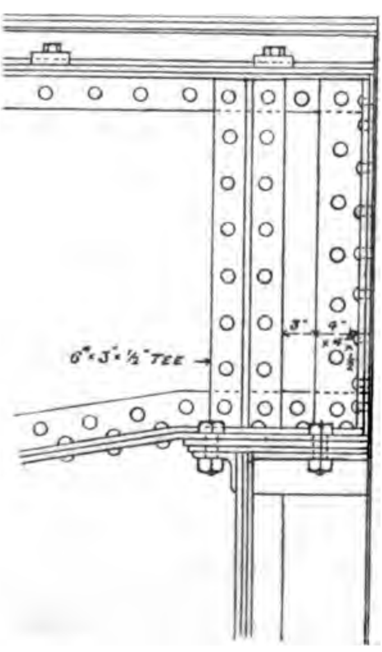


FIG. 70.
Scale ¾ inch = 1 foot.

the curved outline, and the bending of the main angles in the bottom flange, but the extra cost is not very considerable.

It may be remarked in passing that great stiffness or freedom from excessive deflection is frequently of considerable practical advantage in those cases where shafting is attached, as it often is, to traveller girders by means of bracketing, and where any considerable amount of deflection would be detrimental to the action of the shaft. Fig. 68 is a sectional plan at the end of one of the girders showing the seating on the column. The end elevation of the girder, showing the end plate and the bolted connection with the next abutting girder, is shown in Fig. 69, while Figs. 70, 71 show the same detail in side elevation. The joint in the rail is over the joint in the girder, and is covered by a fish-plate in the usual way, allowance being made for expansion. As the joint between two adjacent girders is over a rigid support, there would not appear to be any substantial advantage in this case in making the rail joint to break joint with the girder joint, although cases might arise where such a course would be judicious.

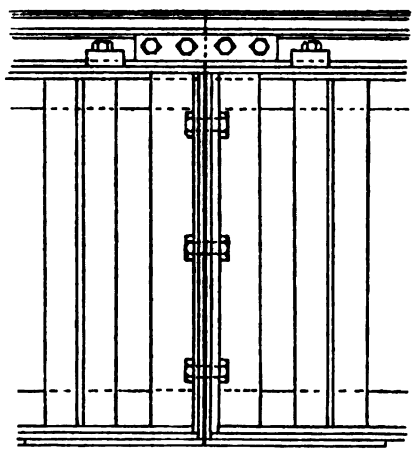


Fig. 71.
Scale ¾ inch = 1 foot.

It frequently happens, in cases where a traveller road has to be supported on one side on a row of columns and on the other side by a wall, that on the wall side special arrangements of girders to carry the rails have to be provided, while their details are determined by the special conditions existing of openings in the walls for light or access, dimensions and spacing of piers, buttresses, or the like.

Examples of some such structure are shown in the accompanying illustrations.

Fig 72 shows in elevation the connection between the ends of two traveller girders, meeting together upon a masonry pier. Fig. 73 shows a cross-section of the girder near the pier, while

Fig. 74 shows the section at the centre of the span. It will be observed that the section of the bottom flange consists of two plates in addition to the main angles, while the top flange at the centre is similar. At the ends, however, while one plate of the

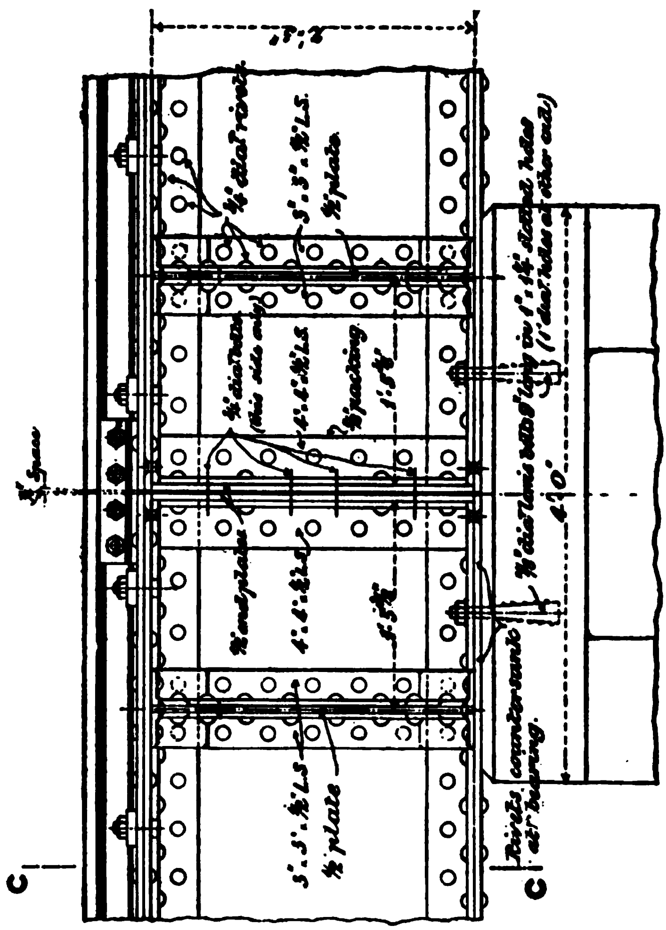


Fig. 72.
Scale ⅛ inch = 1 foot.

bottom flange is stopped off, the two plates in the top flange are run on continuously in order to provide a uniform bed for the rail, the plate continued being in fact a packing, which, had it not been that the clips should be all off one pattern, might for economy sake have been no wider than the rail foot.

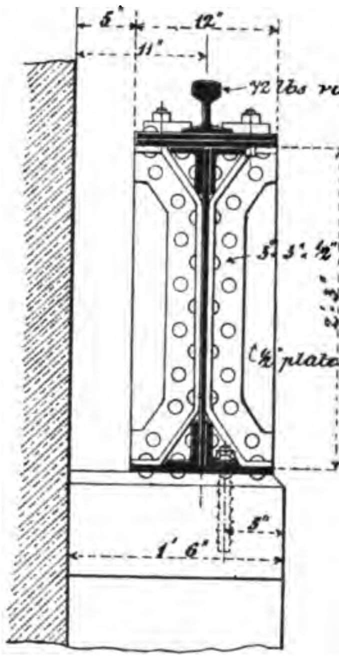


FIG. 73.
Scale ¾ inch = 1 foot.

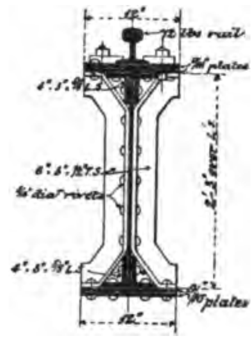


FIG. 74.
Scale ½ inch = 1 foot.

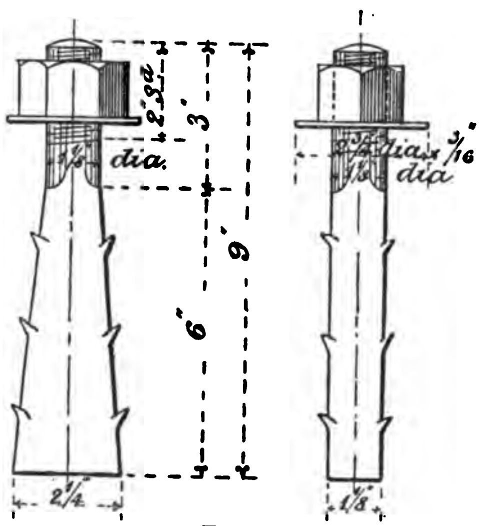


FIG. 75.
Scale 3 inches = 1 foot.

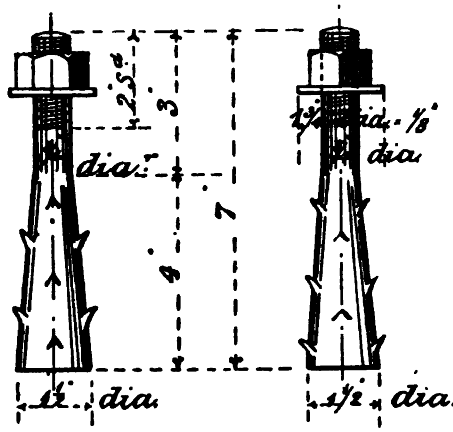


FIG. 76.
Scale 3 inches = 1 foot.

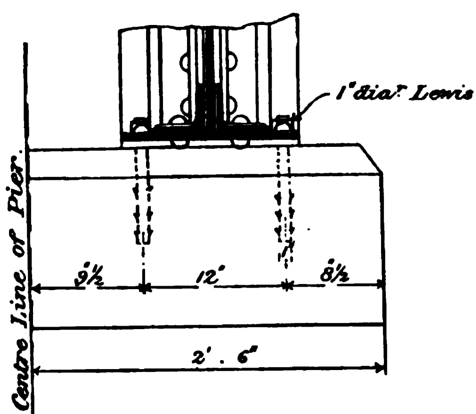


FIG. 77.
Scale ¾ inch = 1 foot.

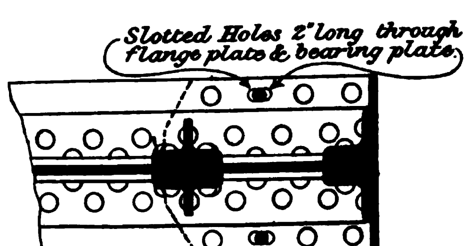


FIG. 78.
Scale ¾ inch = 1 foot.

The important dimension above referred to, viz. the distance between centre line of traveller rail and the adjacent wall is in this case 11 inches. This leaves a net distance of 5 inches between the girder and the wall, and, to avoid any trouble in getting at the work, the lewis bolts securing the girder to the cap stone of the pier are shown on the outer side only; these bolts

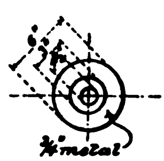


Fig. 79.
Scale ¾ inch = 1 foot.

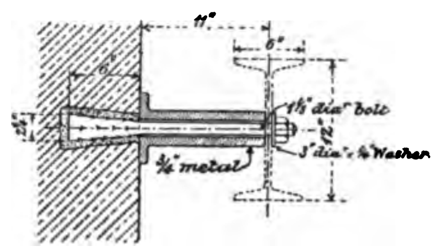


Fig. 80.
Scale ¾ inch = 1 foot.

consisting of ⅞-inch diameter lewis or rag bolts. Similar bolts of ¾-inch and 1⅛-inch diameter are shown on a larger scale in Figs. 75 and 76.

The seating of the girder on a pier at an end wall is shown in Figs. 77, 78. Figs. 79, 80 show a method of affording lateral stiffness to a rolled joist traveller girder alongside a wall.

The important subject of the design and details of lofty columns supporting systems of traveller roads, etc., will be found dealt with in Chapter IV., on the Practical Design of Columns and Struts, where examples of such columns will be found.

Lattice Girderwork for Roofing.—As an example of this application of girderwork, details will now be given of a lattice girder of

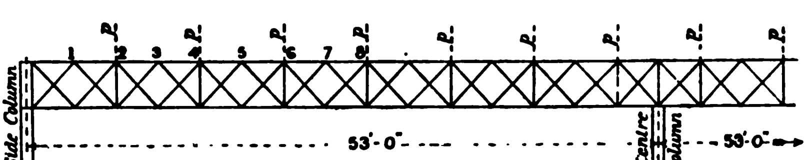


Fig. 81.
Scale 1 inch = 16 feet.

53′ span supporting a series of roof principals, the details of

these principals being given in Figs. 216 and 298 to 307. Fig. 81 gives a skeleton outline of the triangulated girder

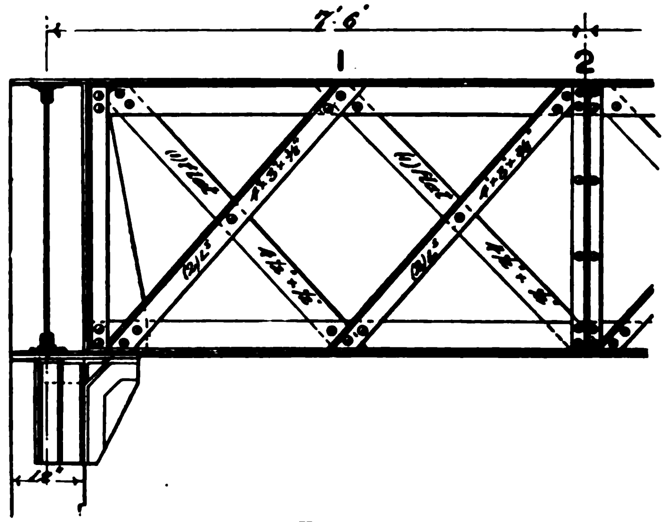


FIG. 82.
Scale ⅜ inch = 1 foot.

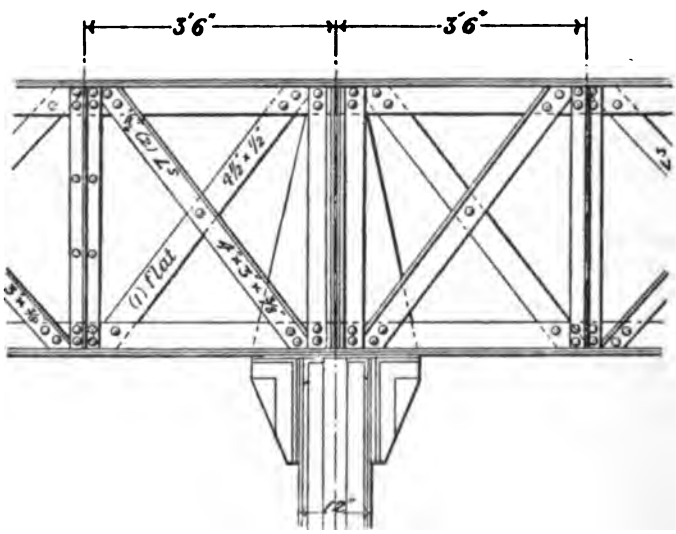


FIG. 83.
Scale ⅜ inch = 1 foot.

the principals being carried immediately over the vertical struts, as shown, *p, p, p.* The girders are carried on side columns and a centre column as shown, but are not continuous, and a roof principal does not occur over the centre column. The general arrangements at side and centre columns are shown in Figs. 82,

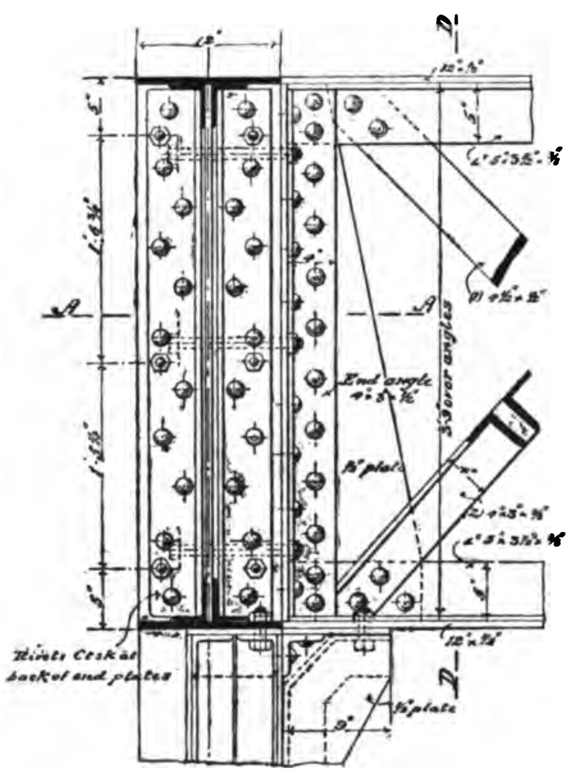


FIG. 84.
Scale ¾ inch = 1 foot.

83, while the details of connections are shown to a larger scale, for the side column in Figs. 84, 85, and 86, the latter being a section of that portion of the column to which the girder is attached, and for the centre column in Figs. 87, 88, 90. The normal section of the girder is shown in Fig. 89. The details

of the apices of the triangulations, or the intersections of the web bracings with the flanges, are shown for apices 2, 4, 6, and 8

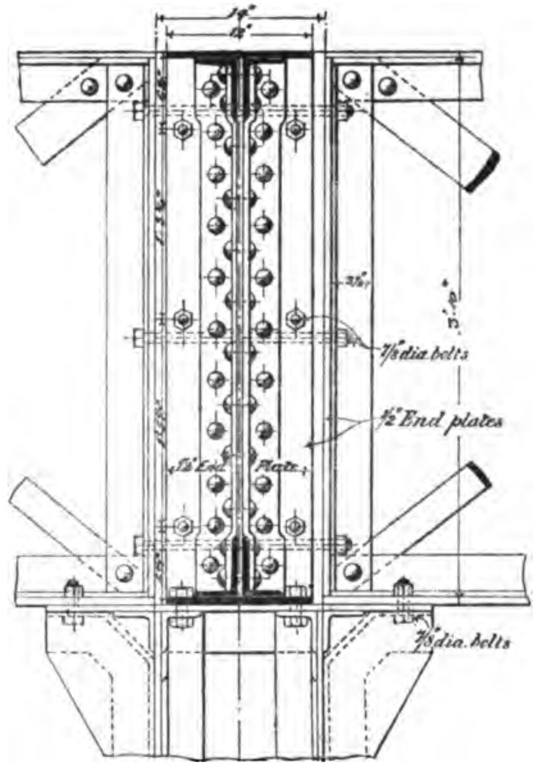


FIG. 85 (Scale ¾ inch = 1 foot).

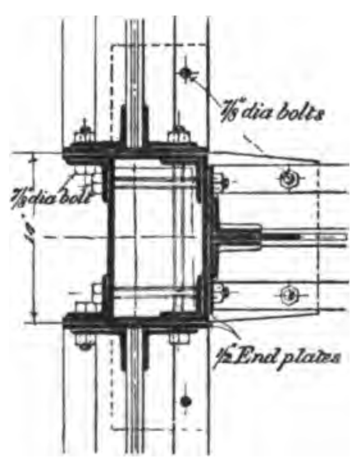


FIG. 86 (Scale ¾ inch = 1 foot).

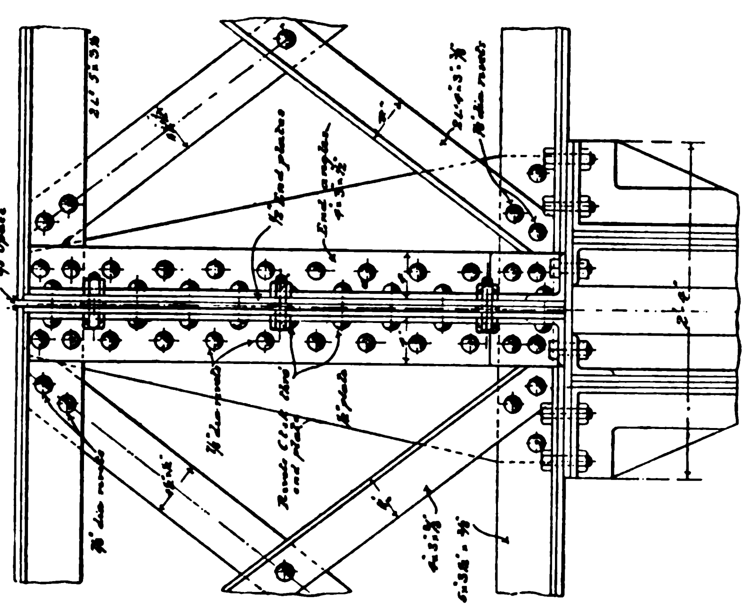


FIG. 87.
Scale ¾ inch = 1 foot.

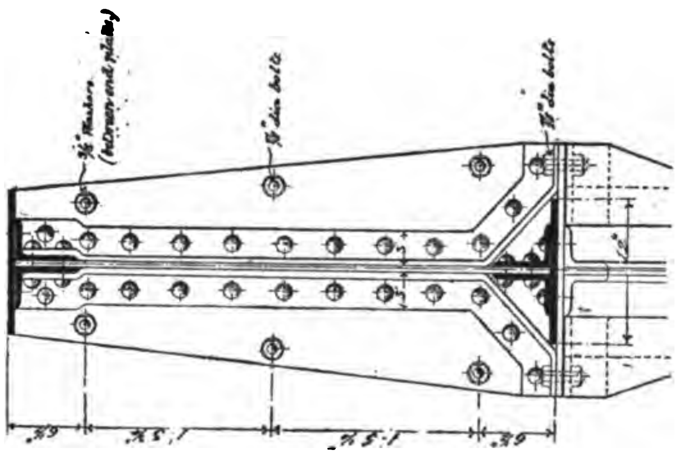


FIG. 88.
Scale ¾ inch = 1 foot.

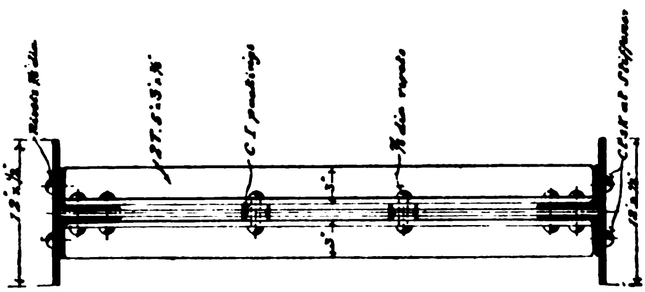


FIG. 89.
Scale ¾ inch = 1 foot.

in Figs. 91, 92, 93, 94, and for apices 1, 3, 5, and 7 in Figs. 95, 96, 97, 98, showing the riveted connections. These are examples

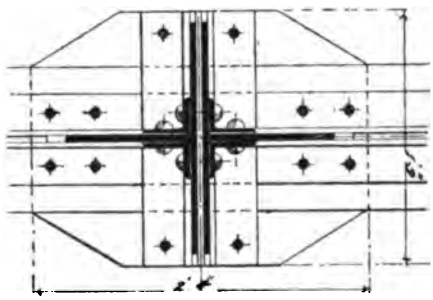


FIG. 90.
Scale ⅜ inch = 1 foot.

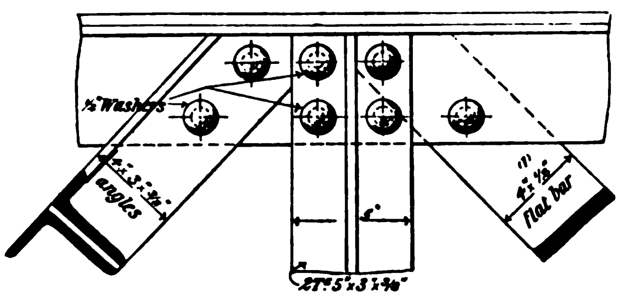


FIG. 91.
Scale 1½ inch = 1 foot.

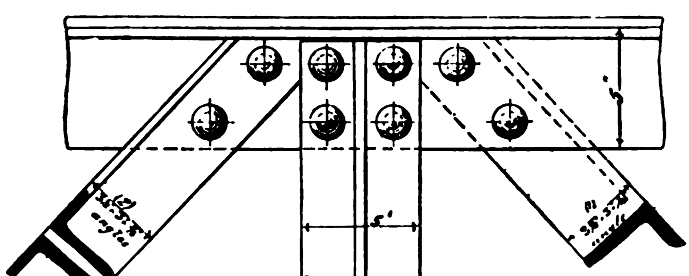


FIG. 92.
Scale 1½ inch = 1 foot.

on a small scale of oblique connections, and may be studied in connection with those for roof-work shown in Chapter IV.

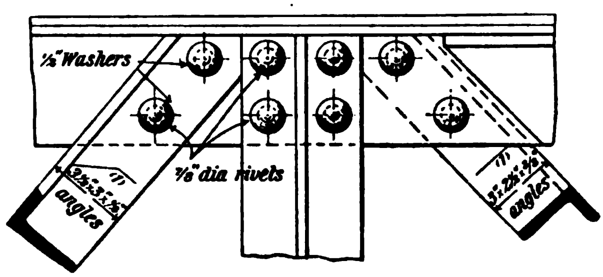


FIG. 93.
Scale 1½ inch = 1 foot.

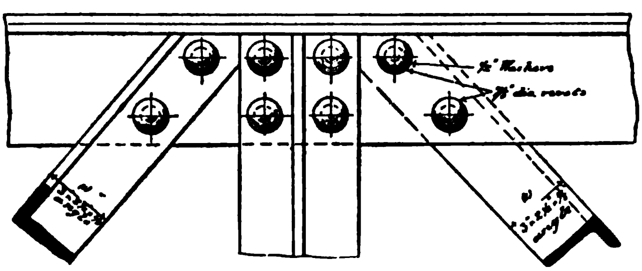


FIG. 94.
Scale 1½ inch = 1 foot.

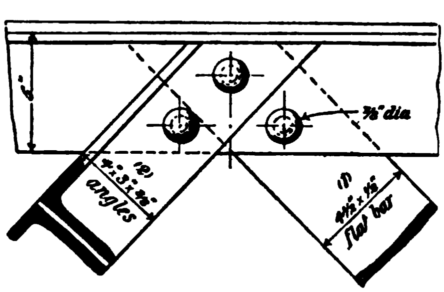


FIG. 95.
Scale 1½ inch = 1 foot.

In Figs. 99, 100, and 101 are given the details of the angle and flange plate joints in the girder, which occur between apices

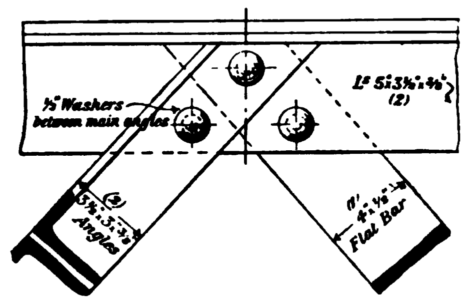


FIG. 96.
Scale 1½ inch = 1 foot.

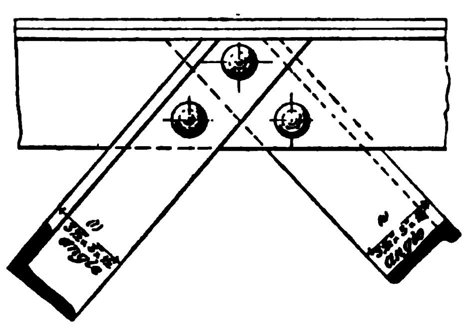


FIG. 97.
Scale 1½ inch = 1 foot.

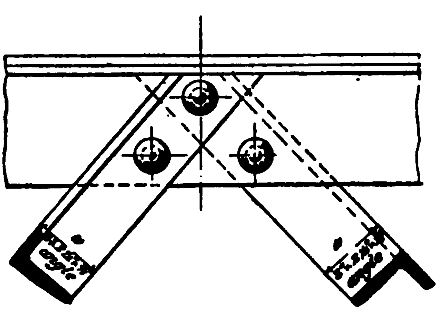
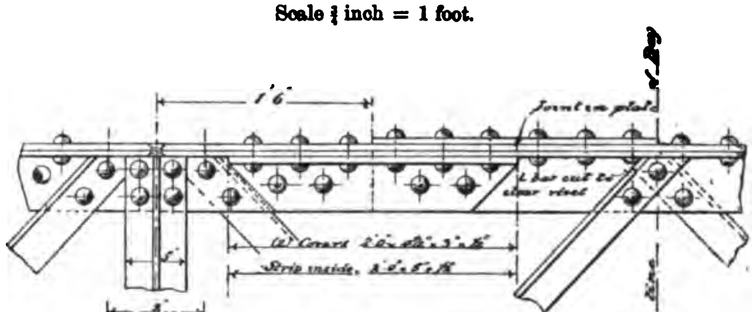


FIG. 98.
Scale 1½ inch = 1 foot.

6 and 7 in the top boom, and break joint in the bottom boom, thus dividing the girder into two lengths for convenience in

FIG. 99.
Scale ¾ inch = 1 foot.



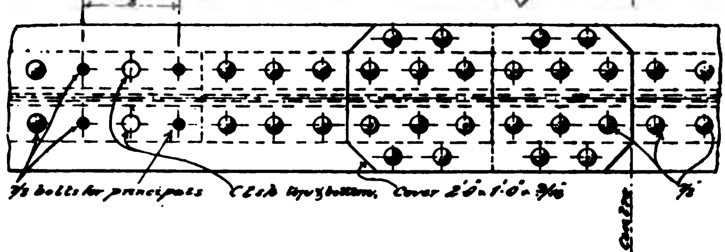


FIG. 100.
Scale ¾ inch = 1 foot.

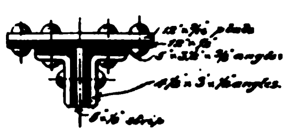


FIG. 101.
Scale ¾ inch = 1 foot.

transport, and keeping the lengths of angles and plates within ordinary limits.

Further details of light lattice girders in connection with roof work will be found in Figs. 326, 337.

Applications of Riveted Girderwork in Connection with Water-tank Construction.—In presenting details of this application of girderwork it has been found advisable to consider at some length (without entering fully upon the important subject of tank construction in general) the details of the tanks themselves, which

in this case are of cast iron, as it is hoped that the practical examples described will not be without value to the student.

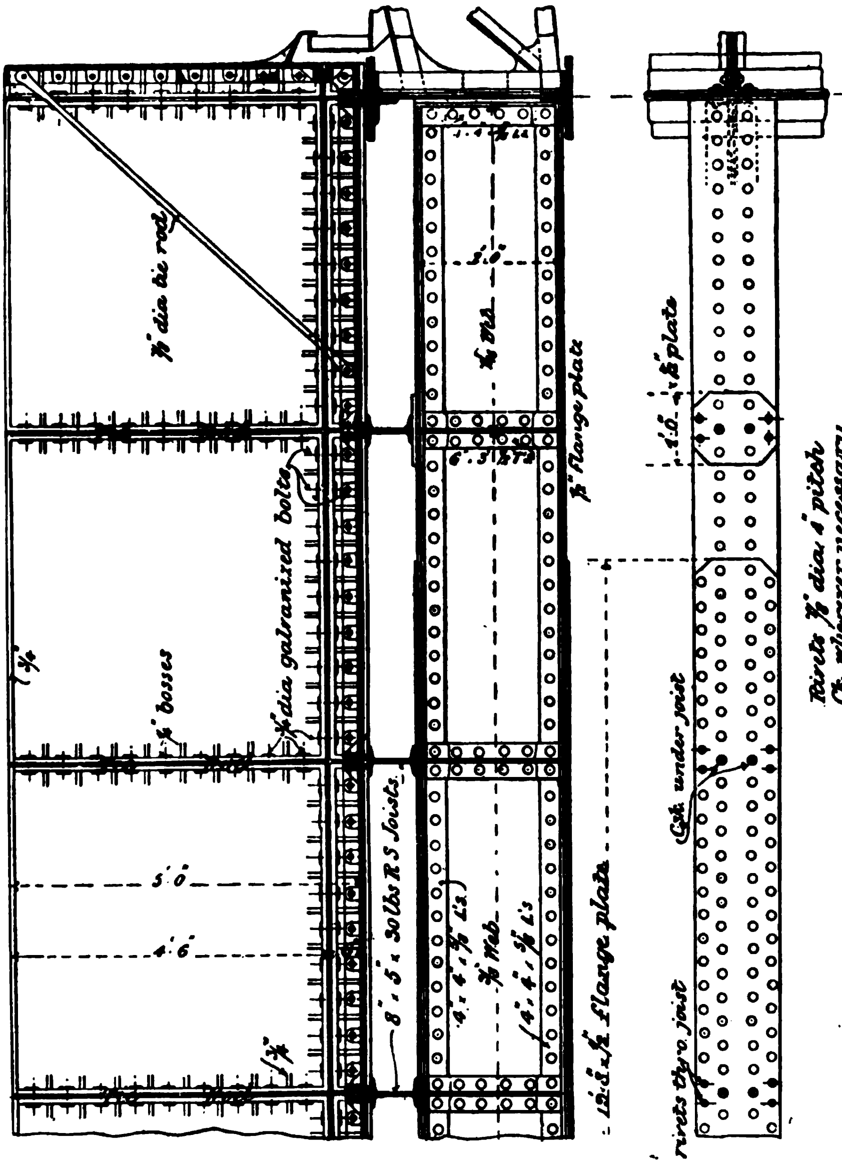


FIG. 102.
Scale ⅜ inch = 1 foot.

Figures 102 to 125 give details of a class of cast-iron tank frequently found in the upper portions of engine houses for pumping machinery, hydraulic accumulators, or other central power stations, and serving as storage tanks for steam or hydraulic machinery, fire purposes, or the like. They not unfrequently form the whole or a portion of the roof in such structures, and where, as in the latter case, they are associated with ordinary roofing details, it is frequently necessary to adopt certain special methods of completing the weathertightness of the whole arrangement.

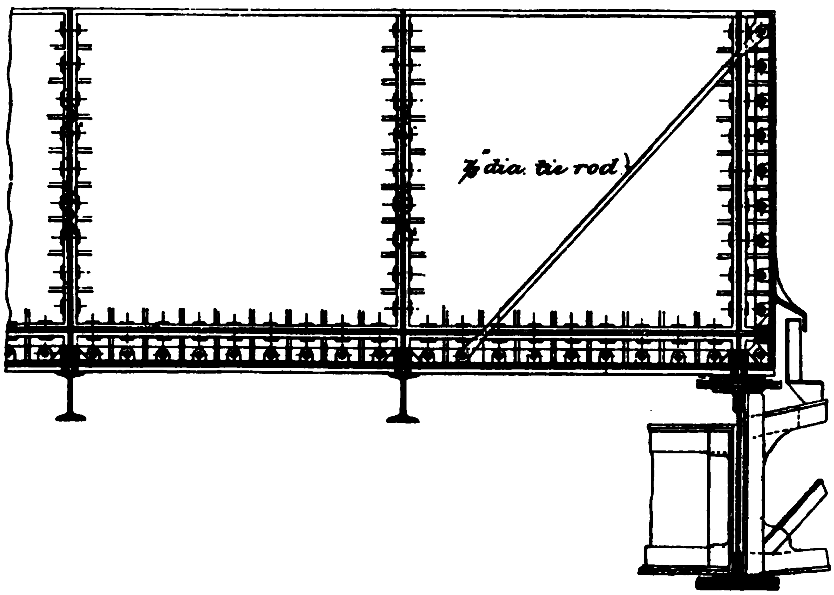


FIG. 103.
Scale $\frac{3}{8}$ inch = 1 foot.

In buildings of any considerable architectural importance it may be desirable to conceal the tank behind the parapet or upper portion of the walls of the main building. This leads to a diminution, other things being equal, of the capacity of the tank, as it is desirable to leave sufficient space between the sides of the tank and the enclosing walls for the purpose of examination, painting, or repairs.

On the other hand, the tank is frequently open to view, and may be treated, as far as possible, as an architectural member of the design, although it must be confessed that the ordinary

methods of embellishment are not usually remarkable for their artistic success.

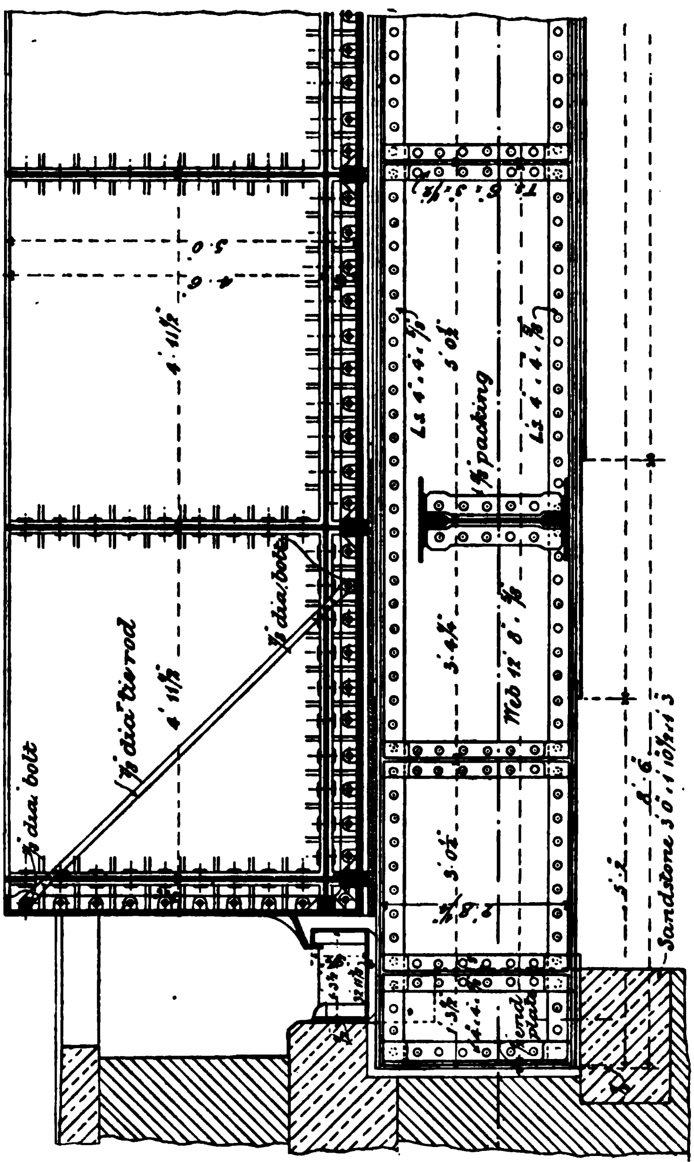


Fig. 104.
Scale $\frac{3}{8}$ inch = 1 foot.

Figures 102, 103, and 104 give part longitudinal and transverse sections of the tank.

Figure 104 shows that side of the tank next to the enclosing parapet wall of the main building. Fig. 102 shows the side of tank next to the roofing and roof principal of special construction referred to in Chapter V., p. 301.

The tank consists of cast-iron flanged plates, forming the

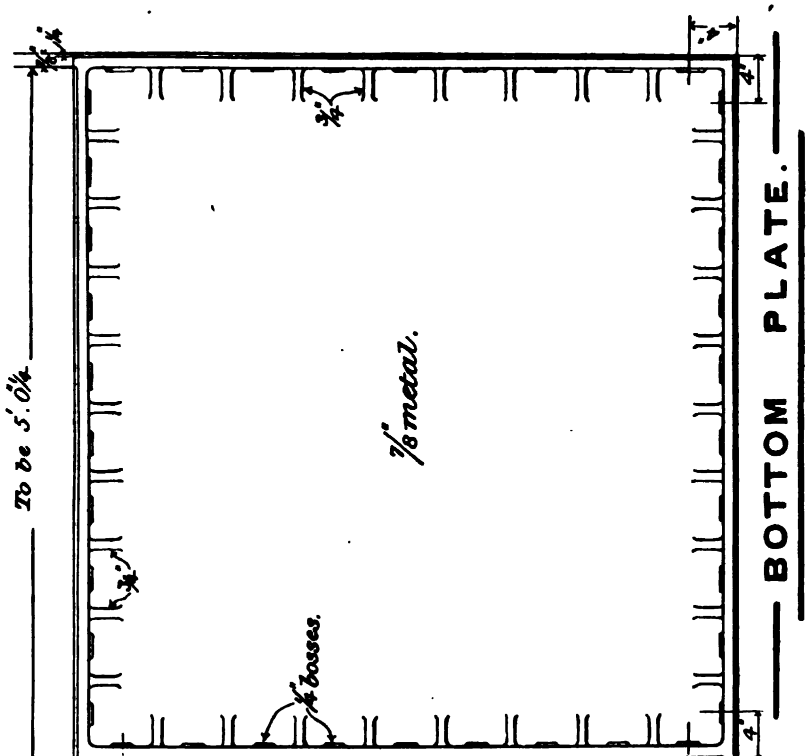


Fig. 106.
Scale ¾ inch = 1 foot.

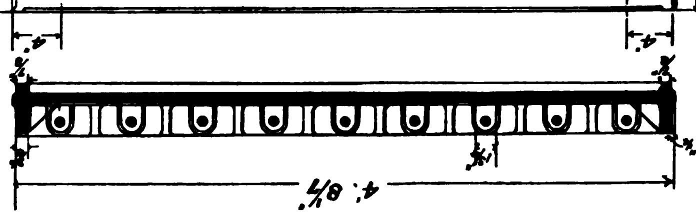


Fig. 105.
Scale ¾ inch = 1 foot.

bottom and sides. The bottom plates in the tank under consideration are of the dimensions shown in length and width, these dimensions being obtained by the setting out of the girderwork carrying the tank, and are probably about as large as plates of

this kind should be made, having regard to the head of water supported (only about 4 feet 6 inches in this case), and to the necessity of uniformity of thickness in the metal, avoidance of flaws, etc.

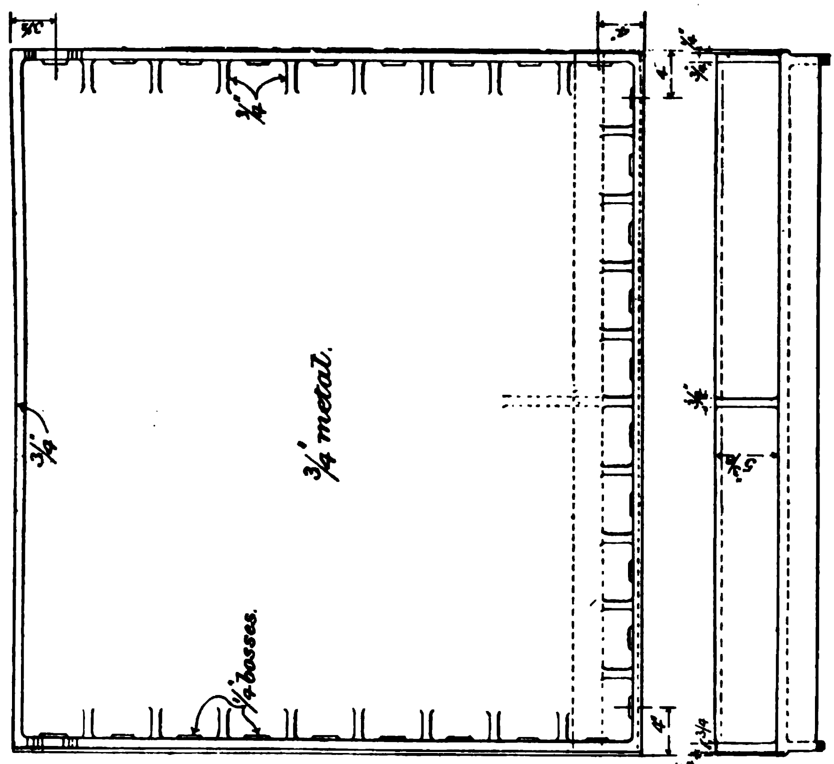


FIG. 108.
Scale ¾ inch = 1 foot.

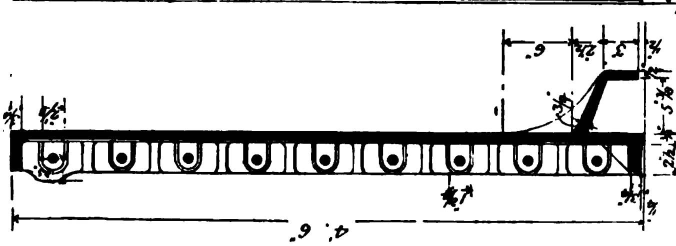


FIG. 107.
Scale ¾ inch = 1 foot.

Bottom and side plates are shown in Figs. 105 to 108.

The side plates are of similar lengths to the bottom plates, and 4 feet 6 inches in height. The thickness of metal in bottom and side plates is $\frac{7}{8}$ inch and $\frac{3}{4}$ inch respectively; the actual

thickness will be governed by the depth of water carried, but the requirements of the foundry will necessitate a limit of thinness beyond which the risk of inequality of metal will be run.

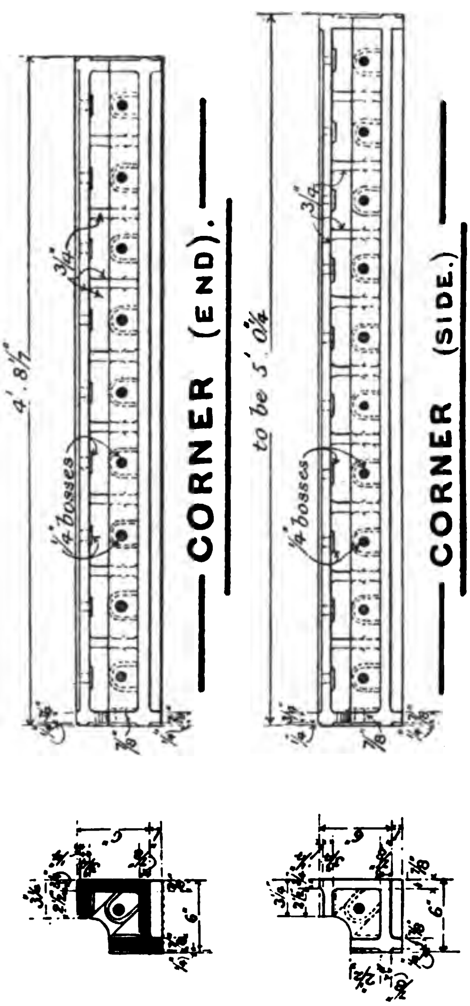


FIG. 109.
Scale $\frac{3}{4}$ inch = 1 foot.

The side plates are connected with the bottom plates by the angle pieces shown in Figs. 102, 103, 104, and on a larger scale in Fig. 109. The angle pieces in this case are shown

with a square corner in the angle. This form, which offers a certain simplicity in the pattern-making, would not be a good one for heavier pressures of water, and an angle or connecting

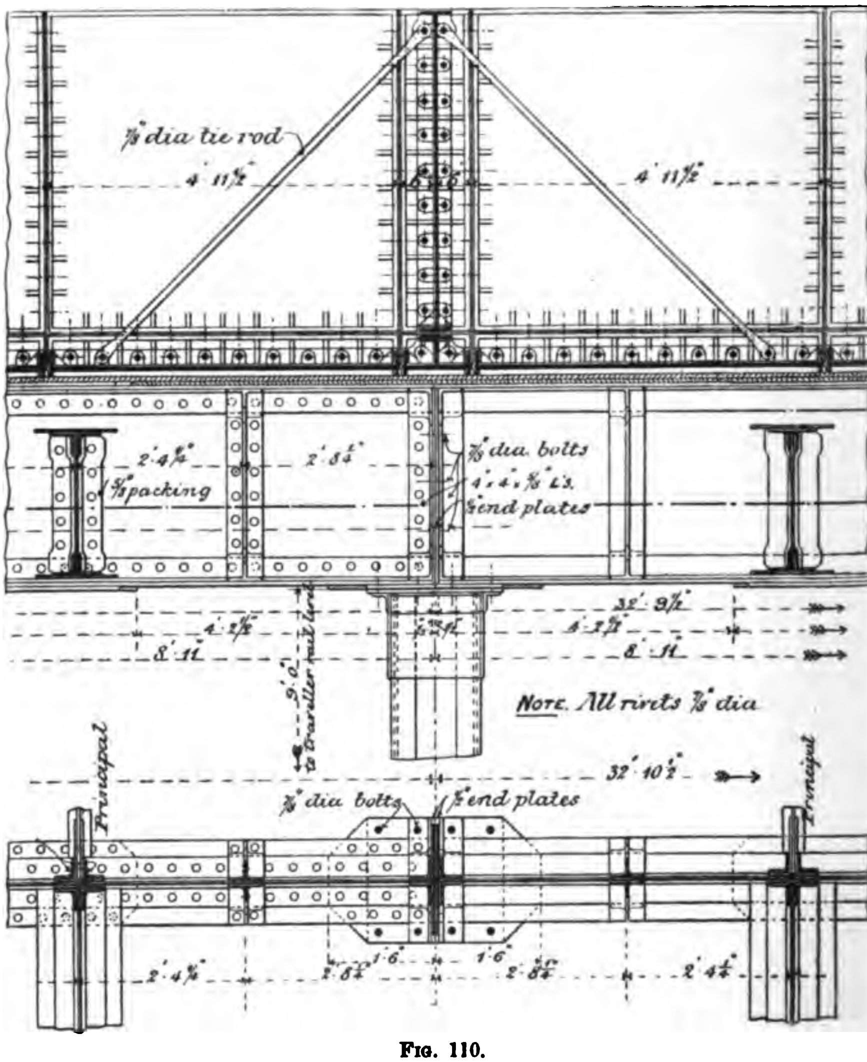


Fig. 110.
Scale ⅜ inch = 1 foot.

piece having a circular quadrant section is frequently used, in accordance with the well-known principle regulating the best form of cast-iron construction under heavy pressures, such, for

example, as the bottoms of hydraulic rams. Such considerations, however, are not to the point in such a case as the present, where the head of water is inconsiderable.

The tank is divided into two halves by a partition of plates similar to the side plates, as shown in Figs. 110 and 111. This division serves the purpose of providing a reserve of water storage when one-half of the tank is laid dry for cleaning or repairs, but it sometimes implies the use of a double set of supply, outlet, and overflow pipes, while the partition itself must be capable of resisting water pressure on alternate sides. The connection of the division plates with the sides and bottom is formed by a

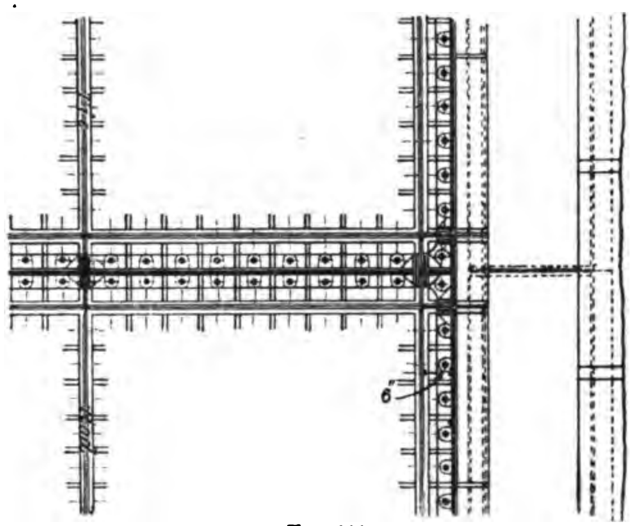


FIG. 111.
Scale $\frac{3}{8}$ inch = 1 foot.

double angle piece, cast in one, as shown in Figs. 110 and 111. The junction at the vertical corners of the tank are also formed by single angle pieces, as shown in plan in Fig. 112.

The whole of the bottom and side plates are provided with flanges round their edges, as shown in the illustrations, of sufficient depth (in this case $2\frac{1}{2}$ inches) to accommodate the size of the bolts used in connection. All the meeting surfaces of these flanges are, in good work, machined where they are in contact, a chipping or planing fillet being provided for that purpose, in such wise that when fitted together a caulking space of about $\frac{1}{2}$ inch in width is left between the flanges, which space is filled up with iron cement

to form a perfectly watertight joint. The flanges are stiffened by a gusset piece between every bolt, the bolt-holes being cored out to receive galvanized bolts. Occasionally the holes are left square, and the bolts provided with square necks. Upon the efficiency of

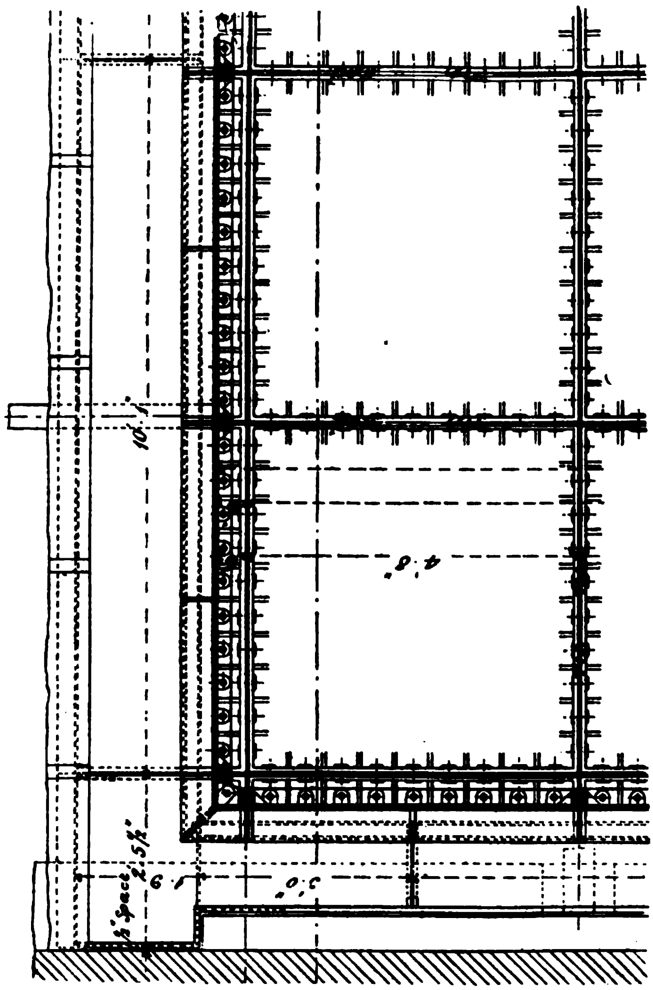


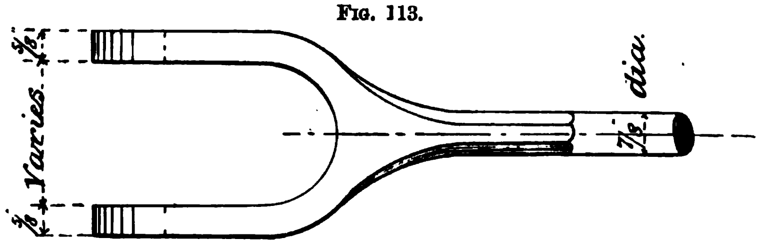
FIG. 112.
Scale $\frac{3}{8}$ inch = 1 foot.

the caulked cement joint the proper watertightness of the tank mainly depends.

It will be observed that, in the tank under consideration, the flanges are turned inside the tank, and not outwards. This is not

an invariable rule, and there are arguments for and against the practice. As regards the strength of the plates, the method shown has the advantage, for experiment has shown that a cast-iron tee-shaped section loaded transversely is stronger with the table downwards than upwards, in accordance with the laws governing the relative resistances in compression and tension of cast-iron sections. On the other hand, the use of the flange turned inwards converts the bottom of the tank into a number of independent pockets without drainage from one to the other, when the tank is laid dry for cleaning purposes. This disadvantage can, however, be met, if necessary, by lining the tank bottom to the level of the top of the flange with Portland cement mortar.

Fig. 113.



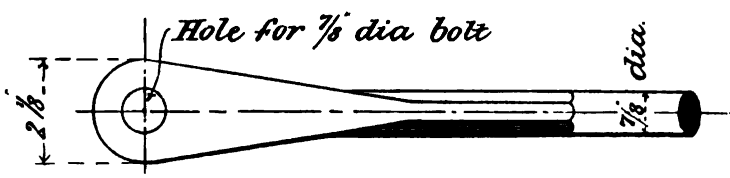


Fig. 114.
Scale 3 inches = 1 foot.

The necessary resistance of the side plates to the bursting pressure of the water is provided in the case under consideration by wrought-iron heavily-galvanized tie-rods, placed at an angle of about 45°, and connecting the top of the side plates with the flanges of the bottom plates. One such rod is provided at every joint in the side, end, and partition plates, as shown in Figs. 102, 104, and 110. These rods are forged with jaws of sufficient width to embrace the pair of flanges at each joint, as shown in Figs. 113 and 114, which show the fork or jaw in plan and elevation. They are bolted at both ends to the flanges of the side and bottom plates, a convenience afforded by the method of turning the flanges inwards. The connection of the tie-rod to the top of the side plates is shown in

Fig. 115. The side plates are further stiffened by a horizontal flange on the upper edge, as shown in Figs. 105 and 115.

In deeper tanks than that shown, the rods are frequently carried horizontally across from one side to the other, at intervals apart depending upon the pressures. In all cases of cast-iron tanks these tie-rods are of vital importance to the security of the

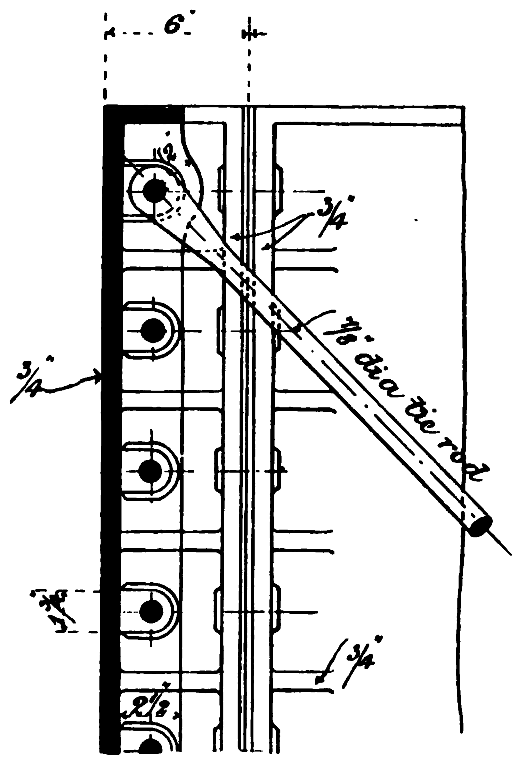


FIG. 115.
Scale 3 inches = 1 foot.

tank, and hardly too much attention can be paid to their design, fitting, and subsequent maintenance.

The total weight of such a tank as that above described, when filled with water, being considerable, careful consideration of the supporting framework of girders is desirable. This framework is shown in Figs. 102, 103, 104, 110, and consists of main riveted plate girders carrying cross riveted plate girders, which last support rolled joists, the whole being of mild steel. The general

arrangement of this girderwork is shown in the figures. The main girders are supported at one end upon the main walls of the building, of which the tank forms a portion of the roof, the other end being carried upon a steel riveted column, which is described in Chapter IV., p. 231. The connection of the main girders over the column is shown in Fig. 110. The cross girders are supported by the main girders, and their connections are shown

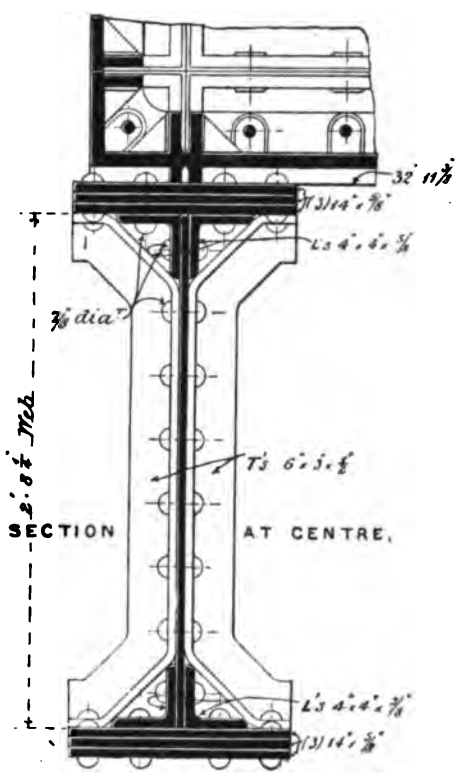


FIG. 116.
Scale 1 inch = 1 foot.

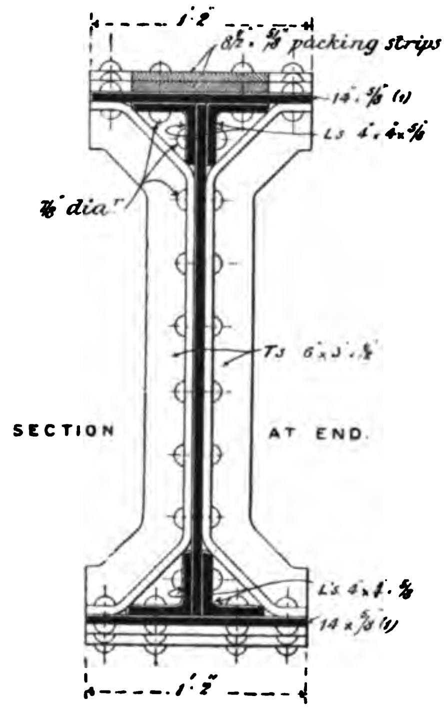


FIG. 117.
Scale 1 inch = 1 foot.

in sectional plan in Fig. 110, and in elevation in Fig. 102. The depths of the main and cross girders are so regulated that the upper surfaces of the rolled joists which rest upon the latter are in the same horizontal plane as the upper surface of the top flange of the main girder. The cross-section of the main girder at the centre is shown in Fig. 116, and at the end in Fig. 117. It will be observed that, as the number of plates in the upper flange fall off

towards the ends, the level is preserved by means of the $8\frac{1}{2}'' \times \frac{5}{8}''$ packing strips, shown in Fig. 117, forming together, with the upper surfaces as aforesaid of the rolled joists, which are carefully straightened and levelled in the press, the plane surface upon which the cast-iron bottom plates of the tank are bedded, the actual contact being made by the 1-inch deep fillets cast on the underside of the plates, as shown in Figs. 103 and 116.

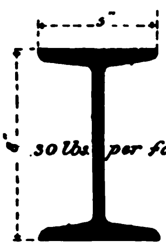


FIG. 118.
Scale $1\frac{1}{4}$ inch = 1 foot.

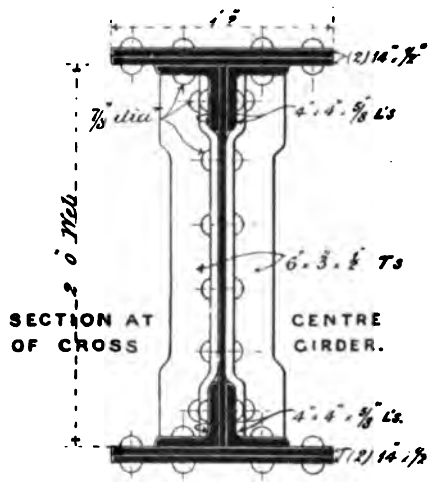


FIG. 119.
Scale 1 inch = 1 foot.

These fillets are chipped or machined until true and even contact is obtained.

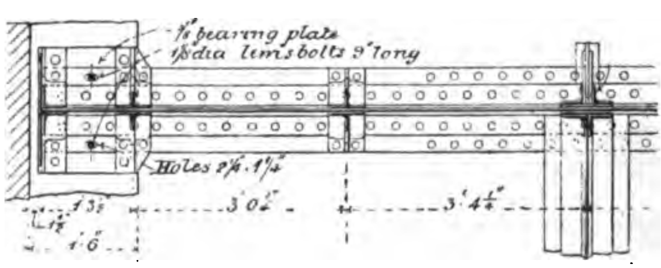


FIG. 120.
Scale $\frac{3}{4}$ inch = 1 foot.

The cross-section of the rolled joist is shown in Fig. 118, and that of the cross girder at the centre in Fig. 119.

The bearing of the main girder on the wall is shown in Fig. 104 in elevation, and in sectional plan in Fig. 120. The bearing on the

column is shown in Fig. 110. The bearings of the rolled joists on the wall are shown in Figs. 121 and 122.

The object sought to be obtained in the arrangement of girderwork above described is to prepare a practically rigid and even bed for a tank constructed of a material (cast iron) not well adapted to resist cross strains arising from excessive or unequal deflection, and

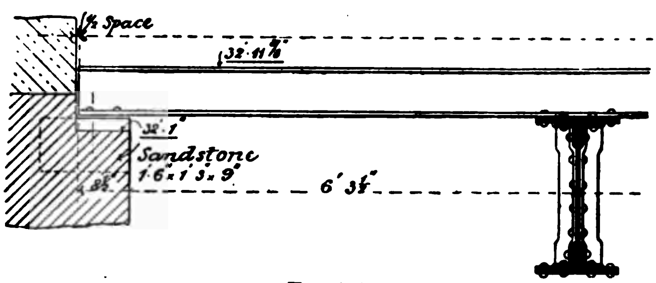


FIG. 121.
Scale $\frac{3}{8}$ inch = 1 foot.

in which it is of prime importance to preserve the joints from starting and becoming leaky.

To prevent undue deflection or alteration in shape arising from the difference between a full and empty tank, it is desirable either to give the supporting girders an ample proportion of depth to

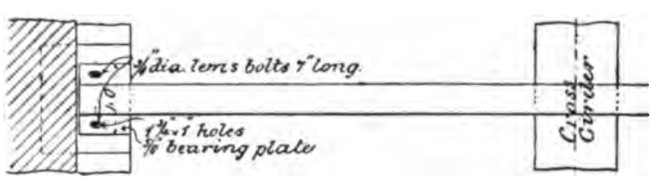


FIG. 122.
Scale $\frac{3}{8}$ inch = 1 foot.

span, if space will allow, or otherwise to keep the working stresses low, by extra metal in the flanges.

It has been above remarked that in cases where tanks of this class form a portion of the roof of the building, and are associated with ordinary roofing details, some special arrangement for forming a weathertight connection between the two becomes necessary.

Certain methods by which this may be effected are illustrated in Figs. 123, 124, and 125.

In Fig. 102 the tank is shown abutting on a roof principal of the type described in Chapter V., p. 302, carrying a cast-iron gutter shown in detail in Fig. 272; and it is necessary to form a watertight connection between this gutter and the side of the tank. A similar detail occurs in Fig. 104, where a gutter runs round between the tank and the parapet wall. In both these cases the object desired is obtained by the use of the special connection shown to a larger scale in Fig. 123, which consists of a weathering

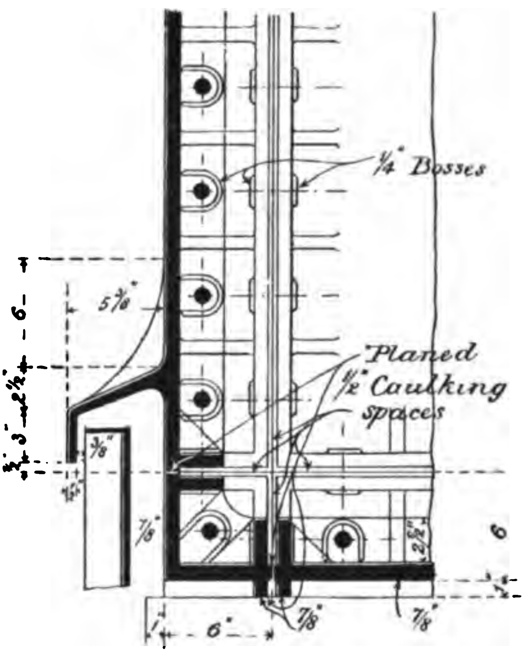


FIG. 123.

flange cast on the outside of the side plates of the tank in the position shown, and having a vertical lip or feather overlapping the gutter, which is tucked in under the flange, as shown. All direct attachment between the tank and the gutter is avoided, either being left free to expand or contract independently of the other. The gutter work is carried completely round the tank as shown in plan in Fig. 112, which also shows the arrangement of the weathering flange at the corners of the tank.

Other methods of attaining the same end are shown in Figs. 124 and 125.

In Fig. 124 a gangway is provided between the tank and the parapet wall for painting or repairs, floored as shown, and covered

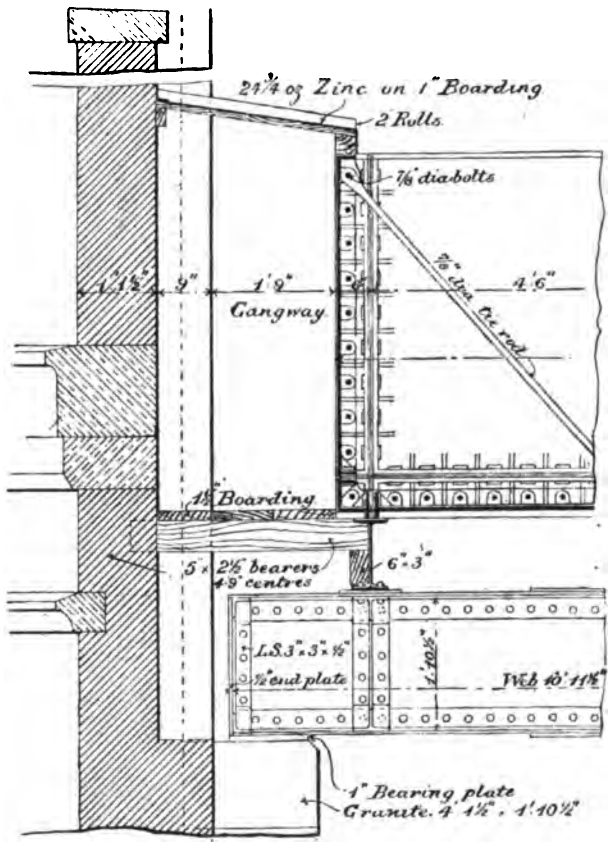


Fig. 124.
Scale ⅜ inch = 1 foot.

with a small lean-to roof of zinc and boarding, draining direct into the tank.

In Fig. 125 connection is made with the gutter of a neighbouring roof in the manner shown, a lining of boarding covered with

zinc being attached to the tank and the gutter in such a way as to prevent wet from getting down into the building.

All the details connected with the hydraulic pipework required in connection with such tanks as those above described cannot here be described. In general three sets of pipes—supply,

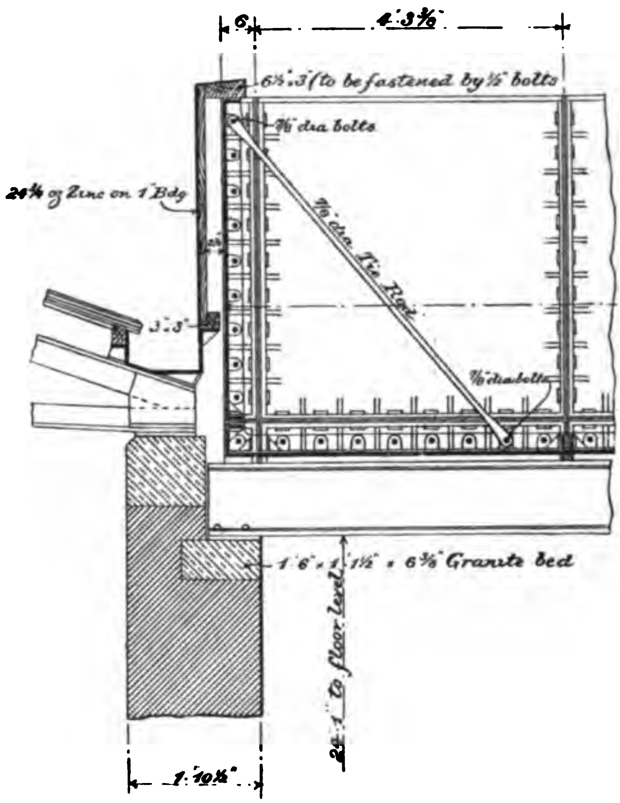


FIG. 125.
Scale ⅜ inch = 1 foot.

discharge, and overflow—will be required, and it is usual to effect the junction of these pipes with the tank by means of special provision in one or more of the bottom or side plates, these being especially stiffened up for that purpose. But in cases where lofty stacks or considerable lengths of pipes are required, the expansion

and contraction of such pipes due to temperature changes should be borne in mind, and not allowed to visit themselves upon the connections to the plates of the tank, with alternating stresses. Automatic or electric tell-tales of water-level will also be required.

Weights of Mild Steel Bolts and Nuts.—The following table of the weights of mild-steel bolts and nuts may be found useful in the process of estimating weights of steel-work and fastenings.

The heads and nuts are hexagonal, of the usual Whitworth standard size; that is, the depth of the head is seven-eighths of the diameter of the bolt, and the depth of the nut is equal to the diameter. The dimensions of the head and nut across the flats or over the angles of the hexagon are those usually given in the published tables of sizes of Whitworth standard.

Any departure from the above proportions of head and nut will of course modify to some extent the weights given in the tables.

The table may also be used for the calculation of weights of similar bolts in other materials than mild steel by proportioning in the ratio of the specific gravities of the material used.

The length of the bolt is in all cases measured from under head to point, and the lengths have been extended to 30 inches in the table to meet the ordinary cases of long bolts used in the connection of heavy timber framing. For longer bolts, such as foundation bolts and the like, special calculations must be made, using the values of the weight of heads and nuts given at the head of the table.

The weight of washers is not included, and must be allowed for separately.

TABLE No. 32.

The Weight of Mild Steel Bolts and Nuts in Pounds per Hundred. Hexagon Heads and Nuts.

Diameter of bolt.	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$
Weight of head and nut together per hundred.	2·83	4·48	7·53	11·71	16·61	22·82	29·4	49·7	74·5
Length in inches under head to point	Weight of bolts and nuts in pounds per hundred.								
1"	4·22	6·65	10·6	16·0	22·2	29·9	38·1	62·2	
$1\frac{1}{4}''$	4·57	7·19	11·4	17·0	23·5	31·6	40·3	65·3	
$1\frac{1}{2}''$	4·92	7·73	12·2	18·1	24·9	33·4	42·4	68·5	100
$1\frac{3}{4}''$	5·27	8·28	13·0	19·1	26·3	35·1	44·6	71·6	104
2"	5·61	8·83	13·8	20·2	27·7	36·9	46·8	74·7	108
$2\frac{1}{4}''$	5·96	9·37	14·5	21·3	29·1	38·7	49·0	77·9	113
$2\frac{1}{2}''$	6·30	9·91	15·3	22·3	30·5	40·4	51·1	81·0	117
$2\frac{3}{4}''$	6·65	10·45	16·1	23·4	31·9	42·2	53·3	84·1	121
3"	7·00	10·99	16·9	24·5	33·3	43·9	55·5	87·2	125
$3\frac{1}{4}''$	7·35	11·53	17·7	25·5	34·7	45·7	57·6	90·4	130
$3\frac{1}{2}''$	7·70	12·08	18·5	26·6	36·1	47·5	59·8	93·5	134
$3\frac{3}{4}''$	8·05	12·63	19·3	27·7	37·5	49·2	62·0	96·6	138
4"	8·39	13·18	20·1	28·7	38·9	51·0	64·2	99·7	142
$4\frac{1}{4}''$	8·74	13·72	20·8	29·8	40·2	52·7	66·3	102·9	147
$4\frac{1}{2}''$	9·09	14·26	21·6	30·9	41·6	54·5	68·5	106·0	151
$4\frac{3}{4}''$	9·44	14·80	22·4	31·9	43·0	56·2	70·7	109·0	155
5"	9·78	15·35	23·2	33·0	44·4	58·0	72·8	112·0	160
$5\frac{1}{4}''$	10·13	15·89	24·0	34·1	45·8	59·8	75·0	115·0	164
$5\frac{1}{2}''$	10·48	16·43	24·8	35·1	47·2	61·5	77·1	118·0	168
$5\frac{3}{4}''$	10·83	16·98	25·5	36·2	48·6	63·3	79·3	121·0	172
6"	11·18	17·53	26·3	37·2	50·0	65·1	81·5	125·0	177
$6\frac{1}{4}''$	11·53	18·07	27·1	38·3	51·4	66·8	83·7	128·0	181
$6\frac{1}{2}''$	11·88	18·61	27·9	39·4	52·8	68·6	85·9	131·0	185
$6\frac{3}{4}''$	12·22	19·15	28·7	40·4	54·2	70·3	88·0	134·0	189
7"	12·56	19·70	29·5	41·5	55·6	72·1	90·2	137·0	194
$7\frac{1}{4}''$	12·91	20·24	30·2	42·6	57·0	73·9	92·4	140·0	198
$7\frac{1}{2}''$	13·26	20·78	31·0	43·6	58·4	75·6	94·6	143·0	202
$7\frac{3}{4}''$	13·60	21·33	31·8	44·7	59·7	77·4	96·7	147·0	206
8"	13·95	21·88	32·6	45·7	61·1	79·1	98·9	150·0	211
$8\frac{1}{4}''$	14·30	22·42	33·4	46·8	62·5	80·9	101·0	153·0	215
$8\frac{1}{2}''$	14·65	22·96	34·1	47·9	63·9	82·7	103·0	156·0	219
$8\frac{3}{4}''$	15·00	23·50	34·9	48·9	65·3	84·4	105·0	159·0	223

Diameter of bolt.	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	3/4"	7/8"
Weight of head and nut together per hundred.	2 83	4·48	7·53	11·71	16·61	22·82	29·4	49·7	74·5
Length in inches under head to point.	Weight of bolts and nuts in pounds per hundred.								
9"	15·34	24·05	35·7	50·0	66·7	86·2	108·0	162·0	228
9 1/4"	15·69	24·59	36·5	51·1	68·1	87·9	110·0	165·0	232
9 1/2"	16·04	25·13	37·3	52·1	69·5	89·7	112·0	168·0	236
9 3/4"	16·39	25·67	38·1	53·2	70·9	91·5	114·0	172·0	240
10"	16·73	26·22	38·9	54·3	72·3	93·2	116·0	175·0	245
10 1/4"	17·08	26·77	39·6	55·3	73·7	95·0	118·0	178·0	249
10 1/2"	17·43	27·32	40·4	56·4	75·0	96·7	120·0	181·0	253
10 3/4"	17·78	27·86	41·2	57·5	76·4	98·5	123·0	184·0	258
11"	18·13	28·40	42·0	58·5	77·8	100·3	125·0	187·0	262
11 1/4"	18·48	28·94	42·8	59·6	79·2	102·0	127·0	190·0	266
11 1/2"	18·83	29·48	43·5	60·7	80·6	103·8	130·0	193·0	270
11 3/4"	19·18	30·03	44·3	61·7	82·0	105·5	132·0	197·0	275
12"	19·53	30·58	45·1	62·8	83·4	107·3	134·0	200·0	279
13"	20·92	32·75	48·3	67·1	89·0	114·4	142·0	212·0	296
14"	22·31	34·92	51·4	71·3	94·5	121·4	151·0	225·0	313
15"	23·70	37·09	54·5	75·6	100·1	128·4	160·0	237·0	330
16"	25·09	39·26	57·6	79·8	105·7	135·5	168·0	250·0	347
17"	26·48	41·43	60·8	84·1	111·3	142·5	177·0	262·0	364
18"	27·87	43·60	63·9	88·3	116·8	149·5	186·0	275·0	381
19"	...	...	67·1	92·6	122·4	156·6	194·0	287·0	398
20"	...	...	70·2	96·8	127·9	163·6	203·0	300·0	415
21"	...	...	73·3	101·1	133·5	170·6	212·0	312·0	432
22"	...	...	76·4	105·3	139·1	177·7	220·0	325·0	449
23"	...	...	79·6	109·6	144·6	184·7	229·0	337·0	466
24"	...	...	82·7	113·9	150·2	191·8	238·0	350·0	483
25"	...	...	...	...	...	...	...	363·0	500
26"	...	...	...	...	...	...	...	375·0	517
27"	...	...	...	...	...	...	...	388·0	534
28"	...	...	...	...	...	...	...	400·0	551
29"	...	...	...	...	...	...	...	413·0	568
30"	...	...	...	...	...	...	...	425·0	585

TABLE OF THE WEIGHT OF MILD STEEL BOLTS AND NUTS IN POUNDS PER HUNDRED. HEXAGON HEADS AND NUTS.

Diameter of bolt.	1"	1⅛"	1¼"	1⅜"	1½"	1⅝"	1¾"	1⅞"	2"
Weight of head and nut together per hundred.	106	148	195	251	324	403	495	600	725
Length in inches under head to point.	Weight of bolts and nuts in pounds per hundred.								
2"	149	204							
2¼"	155	212	273	347					
2½"	161	219	281	357	450	549			
2¾"	167	226	290	368	462	564	682		
3"	173	233	299	378	474	579	699	835	992
3¼"	178	240	308	389	487	593	716	855	1014
3½"	184	247	316	400	500	608	733	875	1036
3¾"	189	254	325	410	512	623	750	894	1058
4"	195	261	334	420	524	638	768	913	1081
4¼"	200	267	342	431	537	653	785	933	1103
4½"	206	274	351	441	549	667	802	952	1125
4¾"	212	281	360	451	562	682	819	972	1148
5"	217	288	368	461	575	697	836	991	1170
5¼"	223	296	377	472	588	712	853	1011	1192
5½"	228	303	386	483	600	726	870	1031	1214
5¾"	234	310	395	493	612	741	887	1050	1236
6"	239	317	404	504	624	756	904	1069	1259
6¼"	245	324	412	515	637	771	921	1089	1281
6½"	251	331	421	526	650	785	938	1109	1304
6¾"	256	338	429	536	663	800	955	1129	1326
7"	262	345	438	546	676	815	972	1148	1348
7¼"	267	352	447	557	688	830	989	1168	1370
7½"	273	359	455	567	700	844	1006	1187	1393
7¾"	278	367	464	578	712	859	1023	1207	1415
8"	284	373	473	588	724	873	1040	1226	1437
8¼"	289	380	481	598	737	888	1057	1245	1459
8½"	295	387	490	609	749	903	1074	1265	1482
8¾"	301	394	499	620	762	918	1091	1284	1504
9"	306	401	508	630	774	932	1108	1304	1526
9¼"	312	408	517	641	787	947	1125	1323	1548
9½"	318	416	525	651	800	962	1142	1343	1571
9¾"	323	423	534	662	812	977	1159	1363	1593
10"	329	430	542	672	825	991	1176	1382	1615
10¼"	334	437	551	683	838	1006	1193	1402	1637

Diameter of bolt.	1″	1⅛″	1¼″	1⅜″	1½″	1⅝″	1¾″	1⅞″	2″
Weight of head and nut together per hundred.	106	148	195	251	324	403	495	600	725
Length in inches under head to point.	Weight of bolts and nuts in pounds per hundred.								
10¼″	340	444	559	694	850	1021	1210	1421	1659
10¾″	345	451	568	704	863	1035	1228	1441	1681
11″	351	458	577	714	875	1049	1245	1460	1704
11¼″	356	465	586	725	887	1064	1262	1480	1726
11½″	362	472	595	736	900	1079	1279	1499	1748
11¾″	367	479	603	746	913	1094	1296	1519	1770
12″	373	486	612	756	925	1108	1313	1539	1793
13″	395	514	646	799	975	1168	1381	1617	1882
14″	418	542	681	841	1024	1227	1449	1695	1971
15″	440	570	716	883	1074	1285	1517	1774	2060
16″	462	598	751	925	1124	1343	1585	1852	2149
17″	484	627	786	967	1175	1403	1654	1930	2238
18″	507	655	821	1008	1225	1461	1722	2008	2327
19″	529	683	855	1051	1274	1520	1790	2087	2416
20″	551	711	890	1093	1325	1578	1858	2165	2505
21″	574	739	925	1135	1374	1637	1926	2243	2595
22″	596	767	960	1177	1425	1696	1994	2321	2684
23″	618	796	995	1219	1474	1754	2062	2399	2773
24″	640	824	1029	1261	1525	1813	2130	2477	2861
25″	662	852	1064	1303	1575	1872	2198	2555	2950
26″	684	880	1098	1345	1625	1931	2266	2633	3039
27″	707	908	1133	1387	1675	1990	2334	2711	3128
28″	729	936	1168	1429	1725	2048	2402	2790	3217
29″	751	964	1202	1471	1775	2107	2470	2868	3306
30″	774	993	1237	1513	1826	2166	2539	2947	3395

CHAPTER IV.

ON THE PRACTICAL DESIGN OF COLUMNS AND STRUTS.

General remarks—The ideal column—The practical column—Variation of modulus of elasticity—Transverse stress: examples—Conditions of end connections: flat ended, round ended, pin ended—Experiments on columns of wrought iron and steel—Wrought-iron rectangular bars and hollow tubes, flat ended—Wrought-iron rectangular bars, pin ended—Influence of size of pins—Tests of wrought-iron riveted columns, flat and pin ended—Table of results—Analysis and remarks—Mode of failure—Weakness at ends of columns—Weakness of component parts of columns—Buckling between rivets—Maximum pitch of rivets compared with plate thickness—Lattice members of columns—Minimum scantlings—Experiments on compressive resistance of various sections—Angles and tees, flat ended—Angles and tees, hinged and round ended—Channels, joists, welded tubes, and Zed columns, flat ended—Channels, joists, and tubes, hinge ended—Wrought-iron latticed columns, pin ended—Mild steel angles, flat ended—Hard steel angles, flat ended—Diagrams of results of formulæ proposed by various authorities—Practical sections of columns and struts—Elementary forms—Flat bars—Angles—Tees—Channels—Channels in combination—Rolled joists—Rolled joists in combination—Built-up sections of various types—Zed-iron sections in combinations—Combinations of channels and joists—Special sections—Phœnix columns—Secondary attachments—Comparison of sections—Relative economy and efficiency—Relative amount of riveting—Relative accessibility for painting—Caution in the preparation of working drawings for columns—Check on proportion of length to diameter—Practical examples of riveted mild steel columns—Procedure with respect to the continuity or otherwise of columns in various floor lengths—Buildings of several stories—Theatre auditorium—Skeleton steel construction in very lofty buildings—Massive columns for engine-house construction carrying travellers and tanks—Variations in type—Columns for machine-shops and engineering works—Complex columns of this type carrying traveller roads and roofing—Foundations to columns—Holding-down bolts—Lateral stability—Special cases for concrete foundations of lofty buildings—Precautions to be observed in the fixing of foundation bolts.

CONSISTENTLY with the principle adopted throughout these notes, the theory of the strength of columns, as viewed from a mathematical standpoint, will not be entered upon. This subject has been frequently dealt with by numerous and able writers, and the

student is referred to their works for further information on this branch of the subject.

The ideal column or strut is perfectly straight, is subjected to a purely compressive stress in the direction of its length, while the compressing force is usually assumed to be truly axial; the modulus of elasticity of the material of which the column is composed is also supposed to be uniform not only in every cross-section, but in every part of a cross-section.

The practical column of everyday experience falls short, however, in a considerable degree from all these ideal conditions, notwithstanding the care with which the designer may have striven to realize them. His column is, it is true, as straight, perhaps, as ordinary workmanship can ensure, but the modulus of elasticity of his material may vary slightly not only in every separate cross-section, but even in different portions of the same cross-section, a physical fact which may, in the life history of the column, determine incipient flexure and perhaps the direction in which that flexure may extend towards the goal of ultimate resistance and failure if the loading be carried to this extent.

So far, again, as the axial direction of loading is concerned, the practical column is often, from the very conditions of the design, exposed to transverse stresses arising either from the bending moment set up by eccentric loading, or even from its own weight, as in the case of inclined struts, such as sheer legs or the jibs of cranes. Vertical columns may also be subject to severe transverse stress where exposed to wind pressure, as, for example, in columns supporting large roofs, or forming the supports of lofty sheds or other buildings. The cast-iron piles of a marine jetty may be instanced as an example where the transverse stresses set up by the force of waves may perhaps be more important than the vertical loading they are called upon to endure. In many cases these transverse stresses have, as far as possible, to be foreseen and allowed for, and the absence of such a provision may, as in the case of crane jibs of considerable radius, have serious results arising from cross strains imposed upon them, let us suppose, by the exigencies of erection or repair.

In addition to the above may be stated the risks of transverse shock, as in the case of columns exposed to wheeled traffic, or to loads piled up against them in such manner as to cause bending stresses in addition to the vertical loading to which they are subjected. In short, it is not too much to assert that the possibilities

of transverse stress in any column should be always present to the mind of the designer, and will go far in guiding his judgment in the determination of that always important point, viz. the ratio of diameter or least dimension to length, and to which further reference will be made.

The ideal column may further be supposed to be either flat ended, round ended, or pin ended, and theoretical deductions have been drawn as to the mode of failure of columns of a certain length under each of these conditions.

But in practice it is not always easy in many cases to assert with confidence under which of the above heads a column or strut should be classed, and, as we shall see in the experiments about to be described, neither pin-ended nor flat-ended columns invariably fail in the mode in which it might be reasoned that they should do.

It is not therefore surprising, from a consideration of the foregoing, to find that most formulæ professing to give the ultimate strength of a column or strut are based upon constants derived from experimental research, although it must be confessed that up to the present time the experimental data available, especially as regards mild steel, do not by any means cover the whole of the ground, or solve all the problems which will present themselves to the designer in the course of his practice.

It is proposed, then, to give in the few pages following a summary in a graphic form of some of the principal experiments on columns of wrought iron and steel—so far as they approximate to the ordinary conditions of practical construction—to be followed by working details of column construction, with explanatory remarks.

In Fig. 126 are plotted the experiments on the compressive resistance of wrought-iron rectangular bars and hollow circular tubes, carried out by Eaton Hodgkinson, and described in the Appendix to the Report of the Commissioners appointed to inquire into the application of iron to railway structures, and carried out in 1846–47.

The rectangular bars varied in length from 3¾ inches to 10 feet, with a sectional area ranging from 1·04 to 5·8 square inches. They were tested in a vertical position, with their ends made perfectly flat and well bedded against two parallel and horizontal crushing surfaces. The proportions of length to least radius of gyration were in several cases extreme, and beyond the

range of practice; as, for example, in the case of a bar 10 feet long and half an inch thick. Ratios beyond the value of 400 to 1

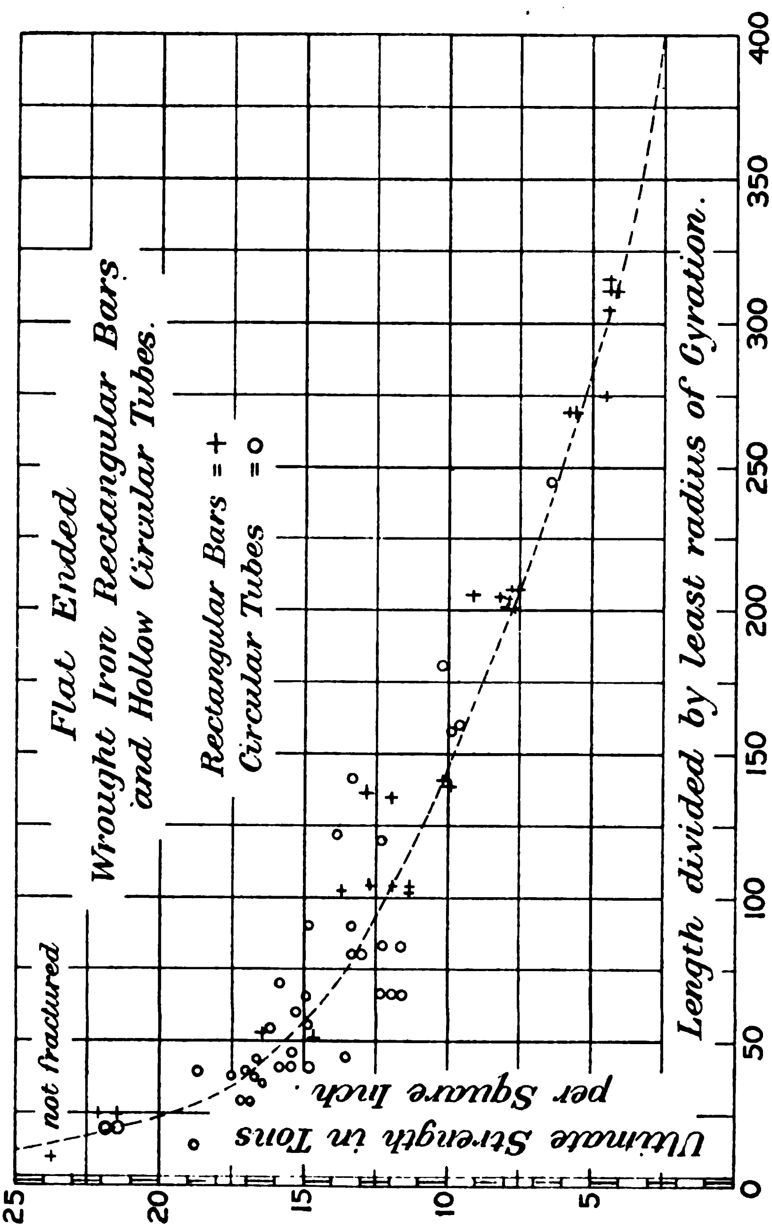


Fig. 126.

TABLE No. 32.

The Weight of Mild Steel Bolts and Nuts in Pounds per Hundred. Hexagon Heads and Nuts.

Diameter of bolt.	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	3/4"	7/8"
Weight of head and nut together per hundred.	2·83	4·48	7·53	11·71	16·61	22·82	29·4	49·7	74·5
Length in inches under head to point	Weight of bolts and nuts in pounds per hundred.								
1"	4·22	6·65	10·6	16·0	22·2	29·9	38·1	62·2	
1¼"	4·57	7·19	11·4	17·0	23·5	31·6	40·3	65·3	
1½"	4·92	7·73	12·2	18·1	24·9	33·4	42·4	68·5	100
1¾"	5·27	8·28	13·0	19·1	26·3	35·1	44·6	71·6	104
2"	5·61	8·83	13·8	20·2	27·7	36·9	46·8	74·7	108
2¼"	5·96	9·37	14·5	21·3	29·1	38·7	49·0	77·9	113
2½"	6·30	9·91	15·3	22·3	30·5	40·4	51·1	81·0	117
2¾"	6·65	10·45	16·1	23·4	31·9	42·2	53·3	84·1	121
3"	7·00	10·99	16·9	24·5	33·3	43·9	55·5	87·2	125
3¼"	7·35	11·53	17·7	25·5	34·7	45·7	57·6	90·4	130
3½"	7·70	12·08	18·5	26·6	36·1	47·5	59·8	93·5	134
3¾"	8·05	12·63	19·3	27·7	37·5	49·2	62·0	96·6	138
4"	8·39	13·18	20·1	28·7	38·9	51·0	64·2	99·7	142
4¼"	8·74	13·72	20·8	29·8	40·2	52·7	66·3	102·9	147
4½"	9·09	14·26	21·6	30·9	41·6	54·5	68·5	106·0	151
4¾"	9·44	14·80	22·4	31·9	43·0	56·2	70·7	109·0	155
5"	9·78	15·35	23·2	33·0	44·4	58·0	72·8	112·0	160
5¼"	10·13	15·89	24·0	34·1	45·8	59·8	75·0	115·0	164
5½"	10·48	16·43	24·8	35·1	47·2	61·5	77·1	118·0	168
5¾"	10·83	16·98	25·5	36·2	48·6	63·3	79·3	121·0	172
6"	11·18	17·53	26·3	37·2	50·0	65·1	81·5	125·0	177
6¼"	11·53	18·07	27·1	38·3	51·4	66·8	83·7	128·0	181
6½"	11·88	18·61	27·9	39·4	52·8	68·6	85·9	131·0	185
6¾"	12·22	19·15	28·7	40·4	54·2	70·3	88·0	134·0	189
7"	12·56	19·70	29·5	41·5	55·6	72·1	90·2	137·0	194
7¼"	12·91	20·24	30·2	42·6	57·0	73·9	92·4	140·0	198
7½"	13·26	20·78	31·0	43·6	58·4	75·6	94·6	143·0	202
7¾"	13·60	21·33	31·8	44·7	59·7	77·4	96·7	147·0	206
8"	13·95	21·88	32·6	45·7	61·1	79·1	98·9	150·0	211
8¼"	14·30	22·42	33·4	46·8	62·5	80·9	101·0	153·0	215
8½"	14·65	22·96	34·1	47·9	63·9	82·7	103·0	156·0	219
8¾"	15·00	23·50	34·9	48·9	65·3	84·4	105·0	159·0	223

Diameter of bolt.	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$
Weight of head and nut together per hundred.	2 83	4·48	7·53	11·71	16·61	22·82	29·4	49·7	74·5
Length in inches under head to point.	**Weight of bolts and nuts in pounds per hundred.**								
9″	15·34	24·05	35·7	50·0	66·7	86·2	108·0	162·0	228
$9\frac{1}{4}''$	15·69	24·59	36·5	51·1	68·1	87·9	110·0	165·0	232
$9\frac{1}{2}''$	16·04	25·13	37·3	52·1	69·5	89·7	112·0	168·0	236
$9\frac{3}{4}''$	16·39	25·67	38·1	53·2	70·9	91·5	114·0	172·0	240
10″	16·73	26·22	38·9	54·3	72·3	93·2	116·0	175·0	245
$10\frac{1}{4}''$	17·08	26·77	39·6	55·3	73·7	95·0	118·0	178·0	249
$10\frac{1}{2}''$	17·43	27·32	40·4	56·4	75·0	96·7	120·0	181·0	253
$10\frac{3}{4}''$	17·78	27·86	41·2	57·5	76·4	98·5	123·0	184·0	258
11″	18·13	28·40	42·0	58·5	77·8	100·3	125·0	187·0	262
$11\frac{1}{4}''$	18·48	28·94	42·8	59·6	79·2	102·0	127·0	190·0	266
$11\frac{1}{2}''$	18·83	29·48	43·5	60·7	80·6	103·8	130·0	193·0	270
$11\frac{3}{4}''$	19·18	30·03	44·3	61·7	82·0	105·5	132·0	197·0	275
12″	19·53	30·58	45·1	62·8	83·4	107·3	134·0	200·0	279
13″	20·92	32·75	48·3	67·1	89·0	114·4	142·0	212·0	296
14″	22·31	34·92	51·4	71·3	94·5	121·4	151·0	225·0	313
15″	23·70	37·09	54·5	75·6	100·1	128·4	160·0	237·0	330
16″	25·09	39·26	57·6	79·8	105·7	135·5	168·0	250·0	347
17″	26·48	41·43	60·8	84·1	111·3	142·5	177·0	262·0	364
18″	27·87	43·60	63·9	88·3	116·8	149·5	186·0	275·0	381
19″	...	...	67·1	92·6	122·4	156·6	194·0	287·0	398
20″	...	...	70·2	96·8	127·9	163·6	203·0	300·0	415
21″	...	...	73·3	101·1	133·5	170·6	212·0	312·0	432
22″	...	...	76·4	105·3	139·1	177·7	220·0	325·0	449
23″	...	...	79·6	109·6	144·6	184·7	229·0	337·0	466
24″	...	...	82·7	113·9	150·2	191·8	238·0	350·0	483
25″	...	...	...	...	...	...	...	363·0	500
26″	...	...	...	...	...	...	...	375·0	517
27″	...	...	...	...	...	...	...	388·0	534
28″	...	...	...	...	...	...	...	400·0	551
29″	...	...	...	...	...	...	...	413·0	568
30″	...	...	...	...	...	...	...	425·0	585

TABLE OF THE WEIGHT OF MILD STEEL BOLTS AND NUTS IN POUNDS PER HUNDRED. HEXAGON HEADS AND NUTS.

Diameter of bolt.	1"	1⅛"	1¼"	1⅜"	1½"	1⅝"	1¾"	1⅞"	2"
Weight of head and nut together per hundred.	106	148	195	251	324	403	495	600	725
Length in inches under head to point.	Weight of bolts and nuts in pounds per hundred.								
2"	149	204							
2¼"	155	212	273	347					
2½"	161	219	281	357	450	549			
2¾"	167	226	290	368	462	564	682		
3"	173	233	299	378	474	579	699	835	992
3¼"	178	240	308	389	487	593	716	855	1014
3½"	184	247	316	400	500	608	733	875	1036
3¾"	189	254	325	410	512	623	750	894	1058
4"	195	261	334	420	524	638	768	913	1081
4¼"	200	267	342	431	537	653	785	933	1103
4½"	206	274	351	441	549	667	802	952	1125
4¾"	212	281	360	451	562	682	819	972	1148
5"	217	288	368	461	575	697	836	991	1170
5¼"	223	296	377	472	588	712	853	1011	1192
5½"	228	303	386	483	600	726	870	1031	1214
5¾"	234	310	395	493	612	741	887	1050	1236
6"	239	317	404	504	624	756	904	1069	1259
6¼"	245	324	412	515	637	771	921	1089	1281
6½"	251	331	421	526	650	785	938	1109	1304
6¾"	256	338	429	536	663	800	955	1129	1326
7"	262	345	438	546	676	815	972	1148	1348
7¼"	267	352	447	557	688	830	989	1168	1370
7½"	273	359	455	567	700	844	1006	1187	1393
7¾"	278	367	464	578	712	859	1023	1207	1415
8"	284	373	473	588	724	873	1040	1226	1437
8¼"	289	380	481	598	737	888	1057	1245	1459
8½"	295	387	490	609	749	903	1074	1265	1482
8¾"	301	394	499	620	762	918	1091	1284	1504
9"	306	401	508	630	774	932	1108	1304	1526
9¼"	312	408	517	641	787	947	1125	1323	1548
9½"	318	416	525	651	800	962	1142	1343	1571
9¾"	323	423	534	662	812	977	1159	1363	1593
10"	329	430	542	672	825	991	1176	1382	1615
10¼"	334	437	551	683	838	1006	1193	1402	1637

Diameter of bolt.	1″	$1\frac{1}{8}$″	$1\frac{1}{4}$″	$1\frac{3}{8}$″	$1\frac{1}{2}$″	$1\frac{5}{8}$″	$1\frac{3}{4}$″	$1\frac{7}{8}$″	2″
Weight of head and nut together per hundred.	106	148	195	251	324	403	495	600	725
Length in inches under head to point.	Weight of bolts and nuts in pounds per hundred.								
$10\frac{1}{2}$″	340	444	559	694	850	1021	1210	1421	1659
$10\frac{3}{4}$″	345	451	568	704	863	1035	1228	1441	1681
11″	351	458	577	714	875	1049	1245	1460	1704
$11\frac{1}{4}$″	356	465	586	725	887	1064	1262	1480	1726
$11\frac{1}{2}$″	362	472	595	736	900	1079	1279	1499	1748
$11\frac{3}{4}$″	367	479	603	746	913	1094	1296	1519	1770
12″	373	486	612	756	925	1108	1313	1539	1793
13″	395	514	646	799	975	1168	1381	1617	1882
14″	418	542	681	841	1024	1227	1449	1695	1971
15″	440	570	716	883	1074	1285	1517	1774	2060
16″	462	598	751	925	1124	1343	1585	1852	2149
17″	484	627	786	967	1175	1403	1654	1930	2238
18″	507	655	821	1008	1225	1461	1722	2008	2327
19″	529	683	855	1051	1274	1520	1790	2087	2416
20″	551	711	890	1093	1325	1578	1858	2165	2505
21″	574	739	925	1135	1374	1637	1926	2243	2595
22″	596	767	960	1177	1425	1696	1994	2321	2684
23″	618	796	995	1219	1474	1754	2062	2399	2773
24″	640	824	1029	1261	1525	1813	2130	2477	2861
25″	662	852	1064	1303	1575	1872	2198	2555	2950
26″	684	880	1098	1345	1625	1931	2266	2633	3039
27″	707	908	1133	1387	1675	1990	2334	2711	3128
28″	729	936	1168	1429	1725	2048	2402	2790	3217
29″	751	964	1202	1471	1775	2107	2470	2868	3306
30″	774	993	1237	1513	1826	2166	2539	2947	3395

26 feet 8 inches in length, four being tested as flat ended and two pin ended.

Six columns of a similar section to that shown in Fig. 163, but with flange and web plates $\frac{7}{16}$ inch thick. The lengths similar to those last described. Four were tested as flat ended and two as pin ended.

Six columns of the type of section shown in Fig. 150, and consisting of two 8-inch channels, with open lattice webs of $2'' \times \frac{3}{8}''$ flats, riveted to the flanges of the channels. The lengths of the columns were 13 feet 4 inches, 20 feet, and 26 feet 8 inches, centres of end pins.

Six columns of similar section to that shown in Fig. 150, but with a swelled outline, the distance between the channels at the centre being from $1\frac{1}{2}$ inch to $2\frac{3}{4}$ inches greater than at the ends. The lengths similar to those last described.

Four columns of the section shown in Fig. 150, and consisting of two 10-inch channels, with open lattice webs of $2\frac{1}{4}'' \times \frac{3}{8}''$ flats, riveted to the flanges of the channels. The lengths of the columns were 16 feet 8 inches and 25 feet, centres of pins.

Four columns of similar section to that shown in Fig. 150, but with a swelled outline, the distance between the channels at the end being from $2\frac{1}{2}$ inches to $3\frac{1}{2}$ inches greater than at the ends. Lengths similar to those last described.

Six columns of a special section of the type shown in Fig. 151, but consisting of two 10-inch channels, with an open lattice web of $2\frac{1}{4}'' \times \frac{3}{8}''$ flats on one side, and a solid plate web $13'' \times \frac{3}{8}''$ on the other. The centre of gravity of this section not being the centre of figure, the placing of the pins in the alternate centres showed the effect of eccentricity of loading with results which will be further referred to.

Six columns of a section similar to the last, but consisting of two 8-inch channels, with an open lattice web of $2'' \times \frac{3}{8}''$ flats on one side, and a solid plate web $12'' \times \frac{3}{8}''$ on the other. These also afforded a similar opportunity of comparing the results of concentric and eccentric loading.

Of the above seventy-four columns, sixteen were tested with flat ends, the remainder being tested with pin ends.

The pins were in all cases $3\frac{1}{2}$ inches diameter, the ends of the columns being reinforced with extra plates and closer riveting, in order to provide sufficient bearing area and resistance to the local stresses set up in the neighbourhood of the pin.

The columns were tested horizontally, and were counter-weighted at the middle. Compressions and sets were measured on a gauged length by a micrometer. The load was gradually applied, and the ultimate load recorded was the maximum which the column was capable of maintaining, although considerable distortions may have previously taken place.

The results of the tests are given in the following table:—

TABLE No. 33.

THE ULTIMATE RESISTANCE TO COMPRESSION OF WROUGHT-IRON PIN-ENDED AND FLAT-ENDED COLUMNS.

Number of experiment.	Description of column.	Length centre to centre of pins. Inches.	Length divided by radius of gyration.	Condition of end bearing. P = pin ended. F = flat ended.	Ultimate strength. Tons per square inch.
1	Type section. Fig. 165. Mean sectional area = 9·91 sq. inches	120·2	62·3	P	13·49
2		120·7	62·3	P	14·01
3		180·0	93·2	P	11·23
4		180·0	93·2	P	9·40
5		240·0	124·3	P	8·65
6		240·1	124·3	P	7·24
7	Type section. Fig. 165. Mean sectional area = 15·90 sq. inches	160·1	38·8	P	13·82
8		160·0	66·1	P	14·06
9		240·0	96·7	P	11·80
10		240·0	96·7	P	10·06
11		320·0	129·0	P	8·79
12		320·1	129·0	P	7·84
13	Type section. Fig. 152. Mean sectional area = 11·46 sq. inches	127·9	46·1	F	14·16
14		127·9	46·1	F	14·98
15		180·0	65·0	P	14·43
16		180·1	65·0	P	14·82
17		240·0	86·6	P	13·37
18		240·0	86·6	P	13·03
19	Type section. Fig. 152. Mean sectional area = 17·85 sq. inches	167·8	47·1	F	15·60
20		167·8	47·1	F	15·89
21		240·0	67·4	P	14·32
22		240·0	67·4	P	14·40
23		320·0	81·4	P	11·82
24		320·0	81·4	P	11·29

Number of experiment.	Description of column.	Length centre to centre of pins. Inches.	Length divided by radius of gyration.	Condition of end bearing. P = pin ended. F = flat ended.	Ultimate strength. Tons per square inch.
25	Type section. Fig. 163. Mean sectional area = 9·41 sq. inches	119·9	49·3	P	13·98
26		120·0	49·3	P	14·15
27		180·0	74·0	P	14·13
28		180·0	64·0	P	13·71
29		240·0	98·7	P	12·92
30		240·0	85·4	P	13·34
31	Type section. Fig. 163. Mean sectional area = 15·87 sq. inches	167·9	50·9	F	14·66
32		167·6	50·8	F	15·65
33		247·6	75·0	F	14·73
34		247·8	65·5	F	15·40
35		319·9	96·9	P	12·51
36		320·0	129·5	P	12·46
37	Type section. Fig. 163. Mean sectional area = 21·12 sq. inches.	167·7	51·1	F	15·03
38		167·7	51·1	F	14·76
39		247·6	75·4	F	14·74
40		247·6	75·4	F	15·15
41		320·0	103·2	P	11·50
42		320·1	103·2	P	11·58
43	Type section. Fig. 150. Mean sectional area = 7·71 sq. inches.	159·2	35·2	P	15·14
44		159·27	35·2	P	16·35
45		239·6	53·0	P	15·23
46		239·6	53·0	P	14·91
47		319·9	107·3	P	14·11
48		319·85	107·3	P	13·33
49	Type section. Fig. 150. Mean sectional area = 7·61 sq. inches.	159·9	35·3	P	15·33
50		159·9	35·3	P	14·97
51		239·7	80·4	P	14·91
52		239·7	53·0	P	15·35
53		319·8	107·3	P	13·76
54		319·92	85·3	P	13·73
55	Type section. Fig. 150. Mean sectional area = 12·14 sq. inches.	199·84	50·3	P	15·06
56		200·0	50·3	P	15·48
57		300·0	61·8	P	15·01
58		300·0	61·8	P	14·48

Number of experiment.	Description of column.	Length centre to centre of pins. Inches.	Length divided by radius of gyration.	Condition of end bearing. P = pin ended. F = flat ended.	Ultimate strength. Tons per square inch.
59	Type section. Fig. 150. Mean sectional area = 12·21 sq. inches.	199·25	41·0	P	13·89
60		199·50	41·0	P	14·28
61		300·2	45·2	P	14·65
62		300·15	75·6	P	14·61
63	Type section. Fig. 151. Mean sectional area = 17·40 sq. inches.	300·0		P	11·69
64		300·0		P	12·57
65		300·0		P	7·77
66		300·0		P	7·71
67		307·75		F	15·30
68		307·87		F	14·88
69	Type section. Fig. 150. Mean sectional area = 12·65 sq. inches.	247·94		F	14·58
70		247·94		F	15·12
71		240·25		P	7·25
72		240·0		P	8·58
73		240·25		P	12·35
74		240·25		P	13·66

Among the practical lessons to be deduced from these experiments, we may first observe that when the ratio of length to least radius of gyration exceeded 120 to 1, sudden lateral springing of the column occurred at or very close to the point of ultimate resistance. This may be compared with the similar phenomenon in the case of 3-inch rectangular bars referred to on p. 184.

It might very naturally have been concluded that in pin-ended columns failure by lateral flexure would take place in a plane at right angles to the axis of the pin, especially when that plane coincided with the plane of the least moment of inertia and radius of gyration.

This method of failure did actually occur in some twenty-four cases, but it is worthy of remark that in fourteen cases, although the plane of lateral flexure was at right angles to the axis of the pin, it was also in the plane of the greatest radius of gyration, not the least. It is for this reason that the fourth column of the table is headed, "Length divided by radius of gyration," not *least* radius of gyration, the proportion of $\frac{l}{r}$ being calculated on that radius in the

plane of which failure took place. It is further to be noted that in several cases failure occurred in a plane *parallel* to the axis of the pins. Such cases should be classified under the heading of fixed-ended struts rather than pin-ended, and even under this condition the plane of failure was not invariably that of the least value of r.

It is not difficult to discover from the results of these experiments, and others which will be referred to, the great importance of avoiding structural weakness at the ends of a column which first receive the stress, whether it be a pin-ended, fixed-ended, or a flat-ended connection. The concentration of stress, and what appears to be in some cases of short columns a certain flow of material under extreme loads, may constitute this the weak point in the entire column strength.

In the case now before us the end of the column is strengthened by the addition of three extra eye-plates on each side, forming a total thickness of metal of $3\frac{3}{8}$ inches and a total bearing surface for the $3\frac{1}{2}$-inch diameter pin of 11·81 square inches. The stress through the eye-plate is transmitted to the body of the column by the rivets, which have a collective shearing area of 8·4 square inches. But this column (No. 42 in the table) failed by the shearing of these rivets, which did not fill the holes, the longest rivets having the most clearance.

Another source of weakness was found to be the bending of the pin, when the unsupported length was too great, accompanied in some cases by the elongation of the pin-holes.

Riveted columns of the type now under consideration may be stated to consist of a number of component parts, the nature of those components and their liability to individual failure varying with the details of design of the column.

It is conceivable that the ultimate strength of a column will be determined by that of its component parts, and that the full strength of the column as a whole will not be attained when local weakness of the component is present. An example of local weakness at the ends of a column has already been described, and this view is further confirmed by an analysis of the experiments upon the box-shaped columns of the types shown in Fig. 163. Of thirty experiments carried out on this type, nine failed by the buckling of the plates between the rivets, and some instructive details may be gathered upon the important point of the proper pitch of rivets in a riveted column required to ensure the maximum

resistance, especially when the column is short. The buckled plate may be considered as a fixed-ended rectangular column, subject to compression in the length of the whole column, the length of this subsidiary or component column being the pitch of the rivets. If, further, this subsidiary column is supposed to be subject to a compressive stress per square inch equal in intensity to that sustained by the whole cross-section, we shall then have the relations expressed in the following table, in which are given the pitch of rivets both crosswise and lengthwise, the thickness of plates, *i.e.* the least dimension of the column, the ratio of $\frac{l}{r}$ both of the component and entire columns, and the ultimate strength per square inch, all as derived from the experiments, all the columns excepting the last (No. 38) having failed by buckling the plates between the rivets.

TABLE No. 34.

THE INFLUENCE OF RIVET PITCH ON THE ULTIMATE STRENGTH OF COLUMNS.

Number of experiment.	Pitch of rivets in inches.		Mean thickness of plates.	Value of $\frac{l}{r}$		Ultimate strength. Tons per square inch.	Nature of end connection. P = pin ends. F = flat ends.
	Crosswise.	Lengthwise.		Plate between rivets.	Entire column.		
25	7·5	6	0·22	95	49·3	13·98	P
26	7·5	6	0·26	80	49·3	14·15	P
13	8·0	6	0·26	80	46·1	14·16	F
14	8·0	6	0·26	80	46·1	14·98	F
19	10·5	6	0·28	74	47·1	15·60	F
32	10·25	6	0·31	67	50·8	15·65	F
31	10·25	6	0·32	65	50·9	14·66	F
34	10·25	6	0·33	63	65·5	15·40	F
37	10·25	6	0·44	47	51·1	15·03	F
38	10·25	6	0·49	42	51·1	14·76	F

It will be observed from the above figures that the 6-inch pitch of rivets represented, so far as the buckling of the plates is concerned, a column of greater length in proportion to its least radius of gyration than the column taken as a whole. This preponderance of slenderness is maintained in the first seven

experiments, decreasing as the plates thicken; in the next two the proportions of $\frac{l}{r}$ are nearly alike, and it may have been a matter of uncertainty whether the column would fail as a whole by lateral flexure, or by buckling as they did between the rivets.

In the last case, however, the thickening of the plate to 0·49 inch has altered the relative proportions of $\frac{l}{r}$, and the plate between the rivets is stronger than the column as a whole. Failure takes place, not by buckling of the plate, but by lateral flexure of the whole column, the plates buckling subsequently to the maximum load being attained. The ultimate resistances to buckling may be compared with the results of Hodgkinson's experiments on flat-ended rectangular bars shown in Fig. 126.

The foregoing analysis is subject to the uncertainty which attaches to the influence which the crosswise pitch may have exerted upon the ultimate buckling resistance, but it at least serves to show that the pitch of riveting should not be overlooked in the design of columns, especially those which, by reason of their ratio of $\frac{l}{r}$, may be expected to give a high ultimate resistance.

In the particular cases cited, it is evident that half an inch was practically the minimum thickness of plate required to prevent buckling with a 6-inch pitch; or, *vice versâ*, that 6 inches was the maximum pitch allowable for $\frac{1}{2}$-inch plates, when the ratio of length to least radius of gyration for the entire column was about 50 to 1.

We may now refer to the results of the experiments of the latticed columns, Nos. 43 to 74 inclusive, of the table on page 190, of which the types are shown in Figs. 150, 151. We obtain from this series some light on the important and difficult subject of the minimum section required for the lattice members of columns of this type and dimension; the evidence is, it must be admitted, only of a limited and negative character, but is valuable as far as it goes.

These lattice bars formed a single triangulated system, arranged at an angle of about 60 degrees, and secured to the flanges of the channel irons by one $\frac{5}{8}$-inch or $\frac{3}{4}$-inch rivet, the breadth of the bars being 2 inches and $2\frac{1}{4}$ inches respectively, and the thickness $\frac{3}{8}$ inch. In the absence of any statement in the record of tests to the contrary, we may assume that bars of these scantlings were

sufficient to develop the full strength of columns of the lengths and dimensions shown in the table.

The percentage of weight of material used to resist what may be termed the secondary stresses in a column, to counteract local flexure, to provide sufficient bearing area at the ends, or in other ways to meet local weakness, in comparison with the total net theoretic weight of the section subject to direct compression, must always claim attention in a preliminary estimate of the weight of any column or strut, and will vary in accordance with the special conditions of each case. It is obvious that the reduction to a minimum of the scantlings of such a system of latticing as that just referred to is desirable on economical grounds, although the practical conditions of riveted connections and the thickness judged necessary for resisting corrosion will limit the extent to which such reduction can be carried.

We may now lastly observe from experiments Nos. 63 to 74 inclusive the influence of eccentric loading upon the ultimate strength of the column.

Comparing experiments 63 and 64 with 65 and 66, we have two sets of columns, similar in length, cross-section, and details of ends, and differing only in the position of the point of application of the load. In the former (Nos. 63 and 64) the load is applied at the centre of gravity of the unsymmetrical cross-section, with a mean ultimate resistance of 12·13 tons per square inch. In the latter (Nos. 65 and 66) the load is applied at the centre of figure, 1·60 inches out of the centre of gravity, or, say, one-fifth of the width of the column, with a mean ultimate resistance of only 7·74 tons per square inch.

The reduction of strength due to eccentric loading is therefore in this case 36 per cent.

Again, comparing Nos. 73 and 74 with Nos. 71 and 72, we find a mean reduction of 39 per cent., the eccentric loading being about one-sixth of the width of the column from the centre of gravity of the section.

It is noteworthy that a measurable amount of lateral flexure is observed at a much earlier stage in the history of the test in the case of the eccentric load, and the total loads required to cause a given degree of lateral flexure are much less in the eccentric load than in the case of that applied at the centre of gravity, a result which is in consonance with theoretical requirements.

If we endeavour to trace the influence of the form of cross-

section upon the ultimate strength as between the H-shaped, the box with plate webs, and the box with lattice webs, it is found that there is little difference between the two latter, the lattice-webbed column being practically as strong as the plate-webbed for all values of $\frac{l}{r}$. The H-shape appears, however, to fall short of the ultimate resistances of the other sections, more especially as the value of $\frac{l}{r}$ becomes greater. Further evidence is, however, required.

The results of the pin- and flat-ended experiments are plotted in Fig. 128. It is not surprising, from a consideration of the foregoing remarks, that the average results are to be represented rather by an area than by a mean line, the varying elements of resistance in a built-up column being more likely to show considerable variations in strength than solid rectangular plates or bars, or simple rolled sections.

Comparing pin-ended with flat-ended columns, we find, with nearly equal values of $\frac{l}{r}$, that the pin-ended columns give an ultimate resistance but little less than that of flat ends.

Thus the mean of experiments 31, 32, 37, and 38 is 15·02 tons per square inch, while the mean of Nos. 25, 26, 45, 46, 52, 55, and 56 is 14·88 tons per square inch. It is probable that the size of pins was not without influence in the ultimate resistance of the pin-ended columns, while, on the other hand, the local buckling of the plates in some of the flat-ended specimens probably lowered the resistance of those columns as a whole.

The well-known experiments of Mr. James Christie, M.Am.-Soc.C.E., on wrought-iron and steel struts, are plotted in Figs. 129 to 132, and Figs. 134, 135, but for a complete description of the whole of the details of these valuable series of experiments the student is referred to the original record.[1]

In Fig. 129 are plotted the results of compression tests on flat- and fixed-ended angles and tees, these forms of struts being of the type-sections shown in Figs. 142 and 147. The angles experimented upon ranged in section from $1'' \times 1'' \times \frac{1}{8}''$ to $4'' \times 4'' \times \frac{3}{8}''$, while the lengths of the struts ranged from $5\frac{15}{16}''$ to $15'\ 5\frac{3}{8}''$, giving a proportion of $\frac{l}{r}$ which varied between 14 and 481.

[1] *Transactions of the American Society of Civil Engineers*, vol. xiii., April, 1884.

The tees, of which the experimental compressive results are plotted in the same figure, varied from 1″ × 1″ to 4″ × 4″ in

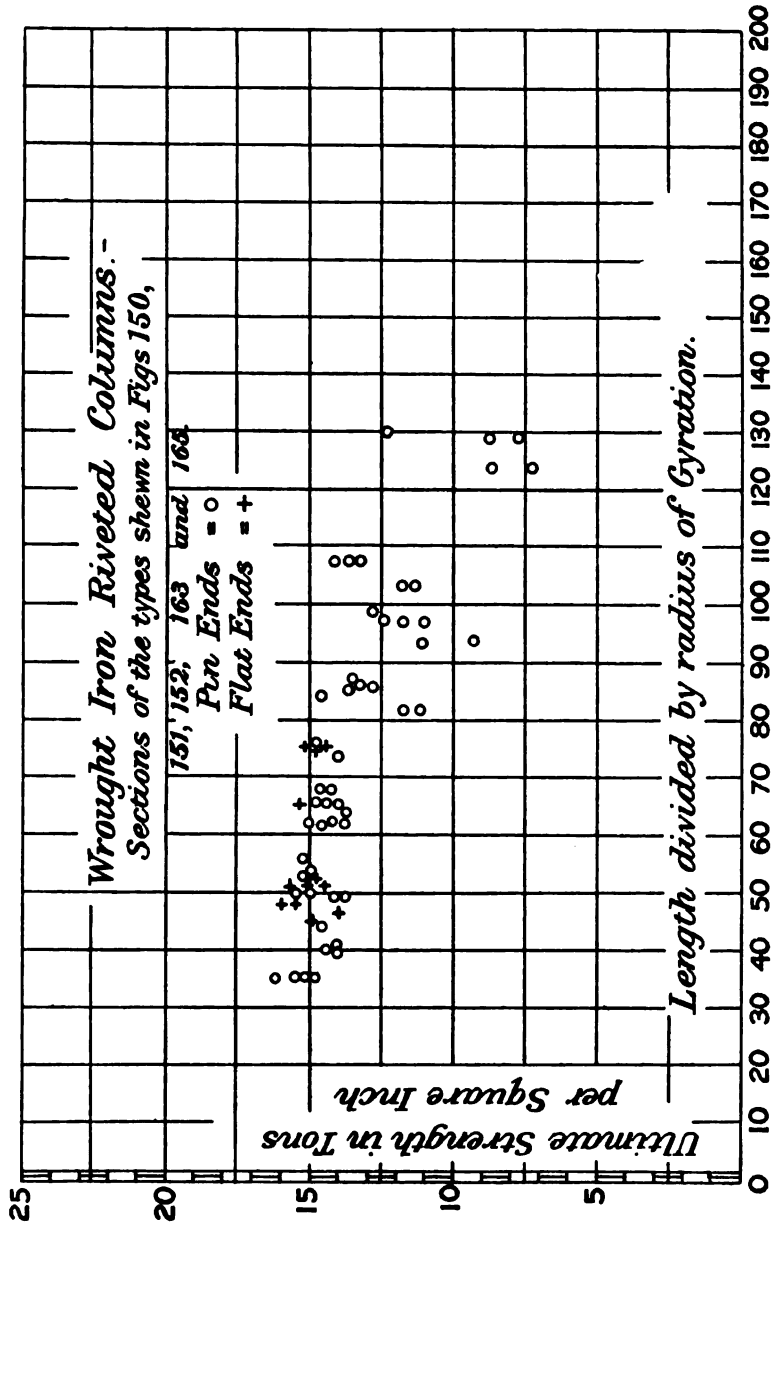


Fig. 128.

section, and from 6″ to 15′ 0⅝″ in length, giving a proportion of $\frac{l}{r}$ ranging between 14 and 420.

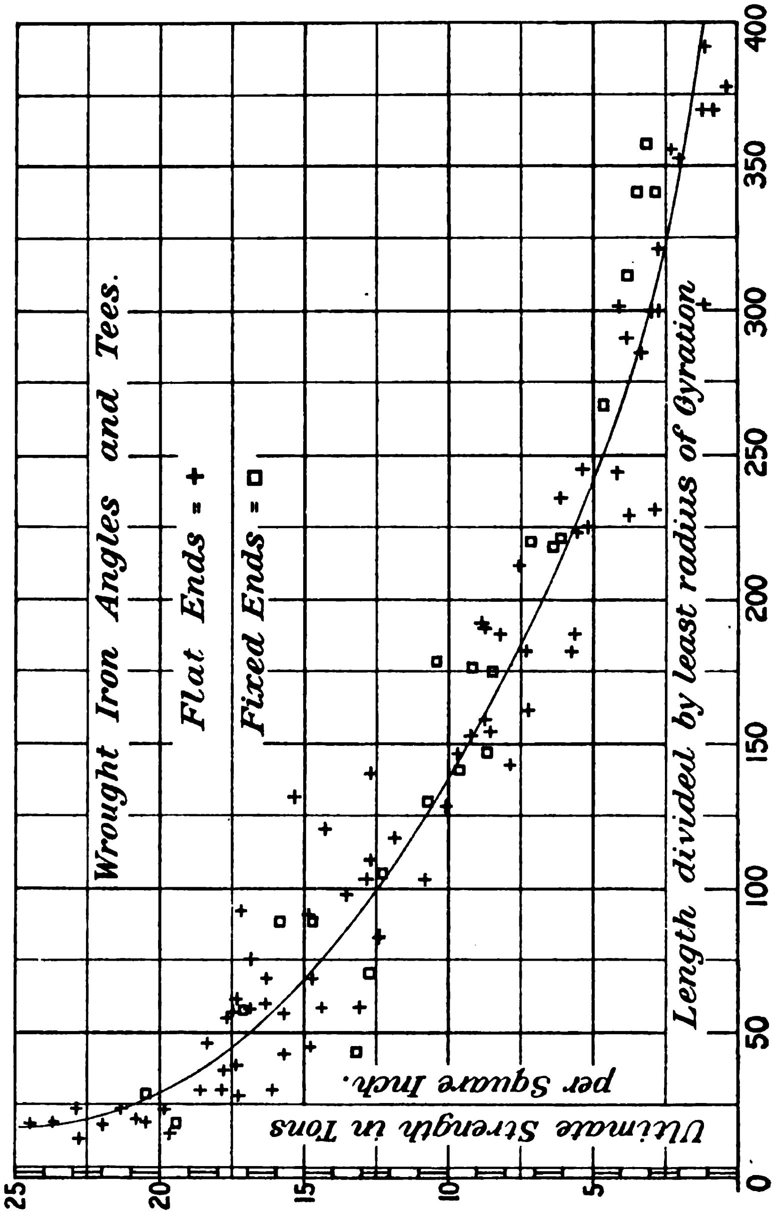


FIG. 129.

The results of experiments on fixed-ended angles are plotted in the same figure. The fixing of the ends was obtained by means of clamps, and it is probable that the theoretical conditions of a fixed-ended strut were more nearly obtained in this series than in flat-ended struts. It will be observed from the diagram that when $\frac{l}{r}$ exceeds about 150, the fixed-ended struts show generally a greater compressive resistance than the flat-ended.

Fig. 130 gives the results of compression tests upon hinged and round-ended angles and tees, both sections being plotted in the same figure. In this case, in the large majority of instances, the hinged ends consisted of ball and socket bearings, a semi-spherical ball of from 1 inch to 2 inches in diameter bearing upon a semi-spherical socket, the specimens being so arranged that the centres of balls were as nearly as practicable coincident with the centre of gravity of the cross-section.

The influence of the size of the ball (probably due, although lubricated, to frictional resistance in the socket) may be traced in one or two instances in this set of experiments.

For example, a $2\frac{1}{2}'' \times 2\frac{1}{2}''$ angle, 5 feet $4\frac{1}{16}$ inches in length, with 2-inch diameter ball, failed at 12·44 tons per square inch, while an angle of the same section and length, with 1-inch diameter ball, failed at 8·18 tons per square inch. Again, an angle $3'' \times 3'' \times \frac{7}{16}''$, 15 feet $3\frac{3}{4}$ inches in length, with 2-inch ball, failed at 2·66 tons per square inch, while the same bar with 1-inch ball failed at 1·31 tons per square inch. Another angle, $2'' \times 2'' \times \frac{5}{16}''$, 15 feet $4\frac{3}{16}$ inches in length, with 2-inch ball, failed at 0·71 ton per square inch, while the same bar with 1-inch ball failed at 0·62 ton per square inch.

The loss of strength produced by slight eccentricity of loading also becomes evident in two tests; an angle, $2'' \times 2'' \times \frac{5}{16}''$, 8 feet $3\frac{1}{8}$ inches long, properly centred, with 1-inch ball and socket ends, failed at 3·16 tons per square inch; improperly centred, it failed at 1·77 tons per square inch. Again, an angle, $1'' \times 1'' \times \frac{1}{8}''$, 5 feet 3 inches long, with 1-inch ball and socket, properly centred, failed at 2·58 tons per square inch; a similar bar, slightly out of centre, failed at 1·30 tons per square inch.

Considerable diversity of results is apparent in the hinged-ended specimens, possibly due to varying frictional resistances between the ball and socket, or pin and bearing, and the results are best expressed by an area, rather than a mean line.

The round-ended specimens were arranged with a semi-spherical ball-bearing on a flat plate. Frictional resistances may be assumed

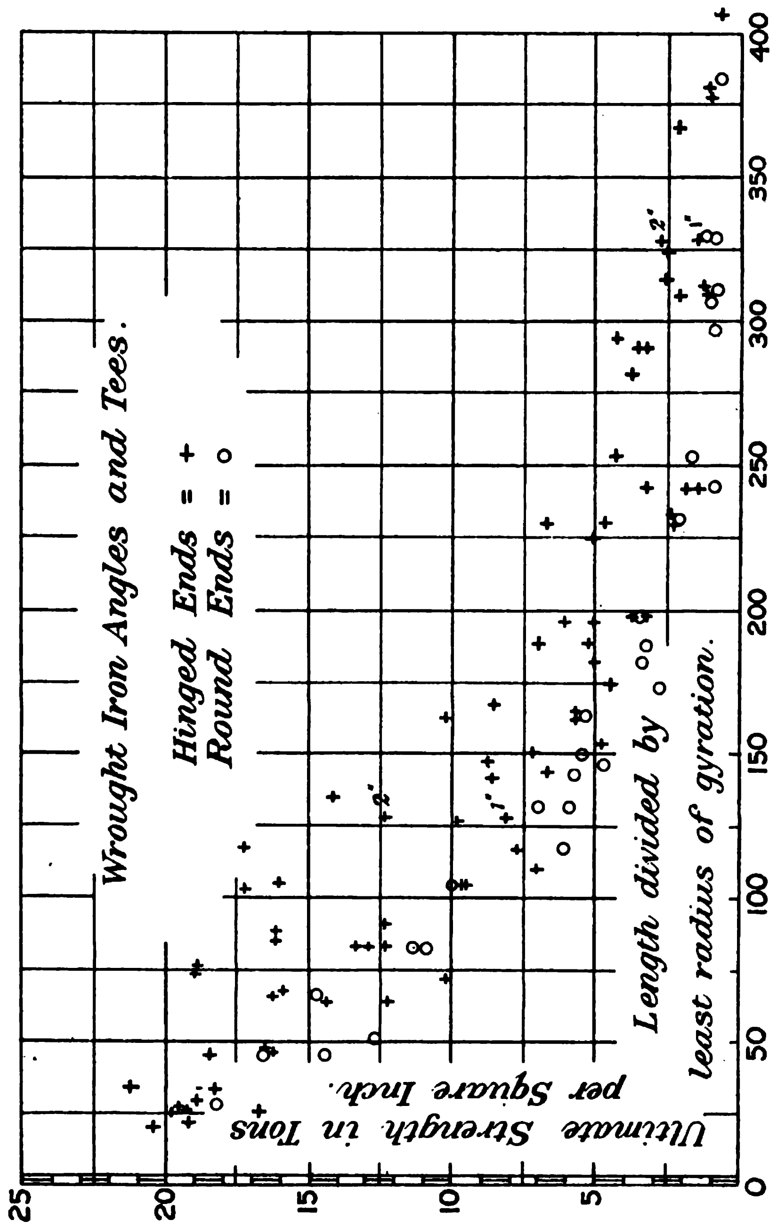


FIG. 130.

to have been absent in this case, and the ends being free to turn upon their bearings, the general results give a somewhat more regular mean line than those of the hinged-ended struts. The line lies, however, obviously along the lower fringe of the hinged-ended area.

Fig. 131 gives the results of compressive tests upon wrought-iron flat-ended channels, joists, welded tubes, and Zed columns. The channels ranged from 2 inches to 12 inches in depth, and from 6 inches to 15 feet in length. The beams ranged from 4 inches to 15 inches in depth, and from 6 feet 6 inches to 22 feet in length.

The tubes were 2·37 and 2·87 inches in outer diameter, and from 3 feet 5 inches to 15 feet in length.

The results of similar sections, but hinged-ended, are shown in Fig. 132. In this series, the influence exerted upon the ultimate strength by the precise conditions of the "hinged-end" are well shown. Thus a welded tube, 3·5 inches external diameter, 15 feet in length, did not fail at 10·5 tons per square inch with a 2-inch pin end, but, with 2-inch ball and socket, failure took place at 6·3 tons per square inch, and with 2-inch ball and plate (round-ended, with practically no frictional resistance) at 5·0 tons per square inch. The result of eccentricity of loading is also shown by a tube of the same diameter and length, with 2-inch pin, set one-tenth of an inch out of centre, which failed at 8·4 tons per square inch.

The quality of wrought-iron used by Mr. Christie in the foregoing tests was as follows: ultimate breaking stress in tension = 21·8 tons per square inch; elastic limit = 14·3 tons; elongation in 8 inches = 18 per cent.

Among the flat-ended specimens plotted in Fig. 131 are shown the results of tests on Z columns with one lattice web carried out by Mr. C. L. Strobel.[1] These tests were fifteen in number, and the section of the column was of the type shown in Fig. 168, consisting of four Z irons, $2\frac{1}{2}'' \times 3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$, connected by a single lattice web. The lengths of the columns ranged from 10 feet $11\frac{1}{4}$ inches to 28 feet. The columns were tested horizontally, the lattice web being in a vertical plane. The ends abutted squarely against the castings of the testing machine, without the interposition of shoes, and the mode of failure was uniformly

[1] *Transactions of the American Society of Civil Engineers*, vol. xviii.

the same, by flexure in the direction of the least radius of gyration, in conformity with theory.

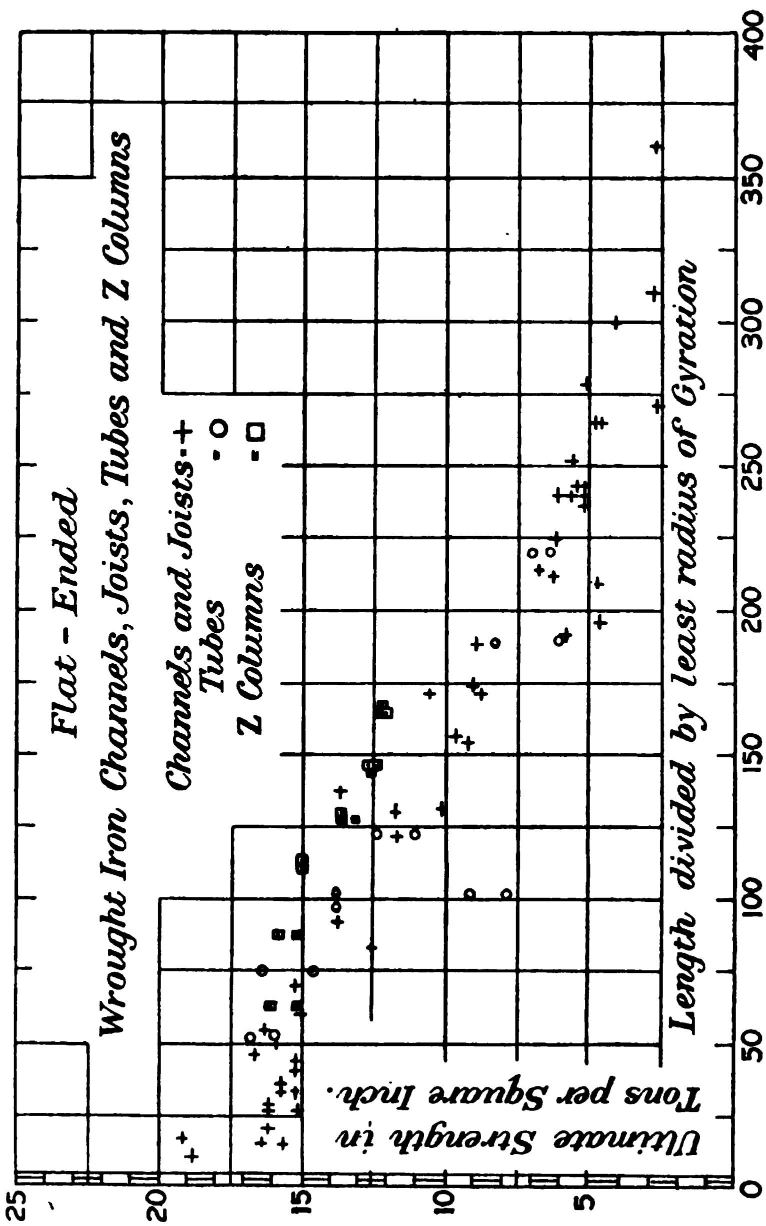


FIG. 181.

The section appears to yield good results, having regard to its simplicity of construction and accessibility for painting. Tensile

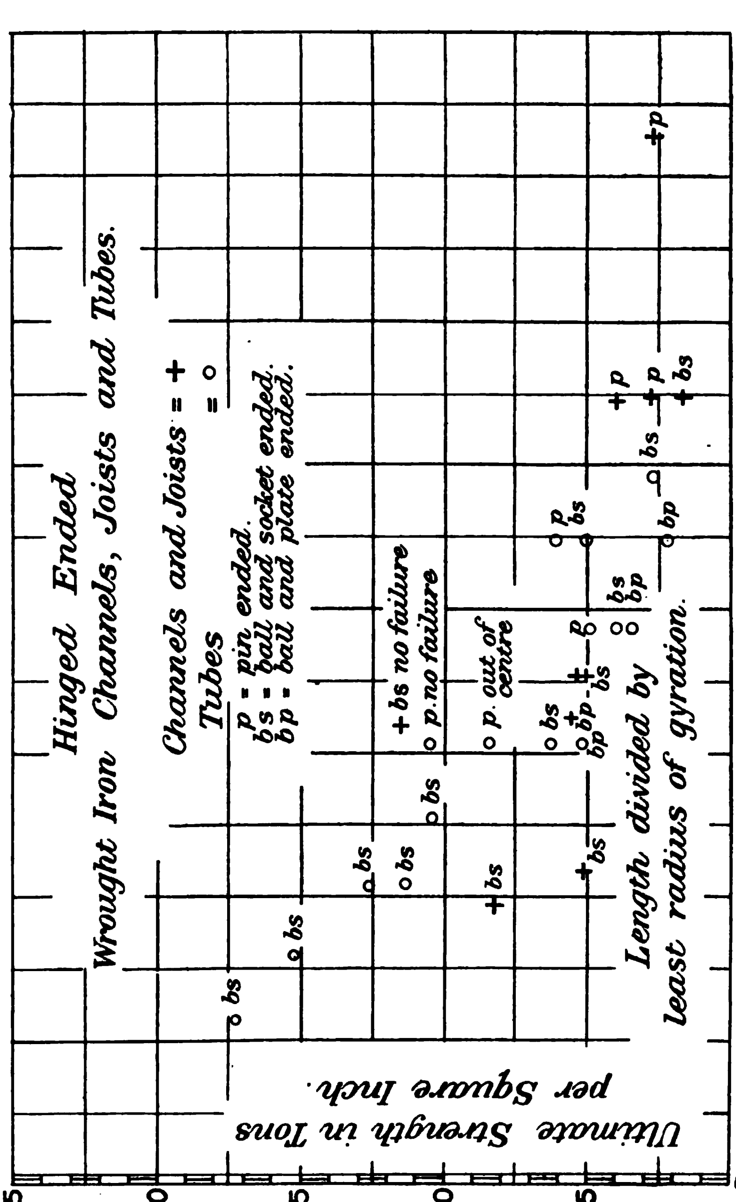


Fig. 132.

tests of the iron used in the Z sections gave an ultimate tensile strength of from 22·0 tons to 24·0 tons per square inch, with an elongation of from 11·3 to 22·7 per cent.

In Fig. 133 are plotted the results of tests upon pin-ended lattice columns of the Detroit Bridge and Iron Company, made at Watertown Arsenal.[1]

These columns were of the type section shown in Fig. 150, consisting of two channel bars, connected together with a double web of lattice bracing. The channels were of sections 6 inches, 8 inches, 10 inches, and 12 inches wide, with flanges of from $1\frac{3}{4}$ inch to $2\frac{1}{2}$ inches deep. The channels were spaced from 6 inches to 8 inches apart, back to back, strengthened with reinforcing plates at the ends to provide bearing for the pin connections, the pins being 3 inches and $3\frac{1}{2}$ inches diameter. The columns failed usually by deflection in the direction of the least radius of gyration.

It will be observed from the figure that 3-inch diameter pins give lower results than $3\frac{1}{2}$-inch diameter, a result in accordance with the tests on 3-inch rectangular bars with pin ends, described on p. 186.

Fig. 134 gives the results in graphic form of Mr. Christie's experiments[2] on flat-ended angles of mild steel, which were carried out under similar conditions to those on wrought iron previously described. The steel employed had an average carbon content of about 0·12 per cent., with an ultimate tensile resistance of about 28·3 tons per square inch, and an ultimate elongation of about 23·6 per cent. in 8 inches.

Fig. 135 gives the results of similar experiments on flat-ended angles of hard steel, having an average carbon content of 0·36 per cent., and an ultimate tensile resistance of about 45 tons per square inch, and an ultimate elongation of $17\frac{1}{2}$ per cent. in 8 inches.

The number of published experiments on the ultimate compressive resistance of mild steel columns is, notwithstanding the extent to which this material has been employed in this direction, not so complete or extensive as those upon wrought-iron columns, and a series of further experiments on full-sized built-up columns of various sections is yet a desideratum.

As an example of such tests, the following may be quoted[3]:—

[1] Lanza, "Applied Mechanics."

[2] *Transactions of the American Society of Civil Engineers*, vol. xiii., August, 1884.

[3] *Ibid.*, vol. xxi.

Two columns of the type shown in Fig. 163, but with the lower flange plate replaced by open latticing, were tested to

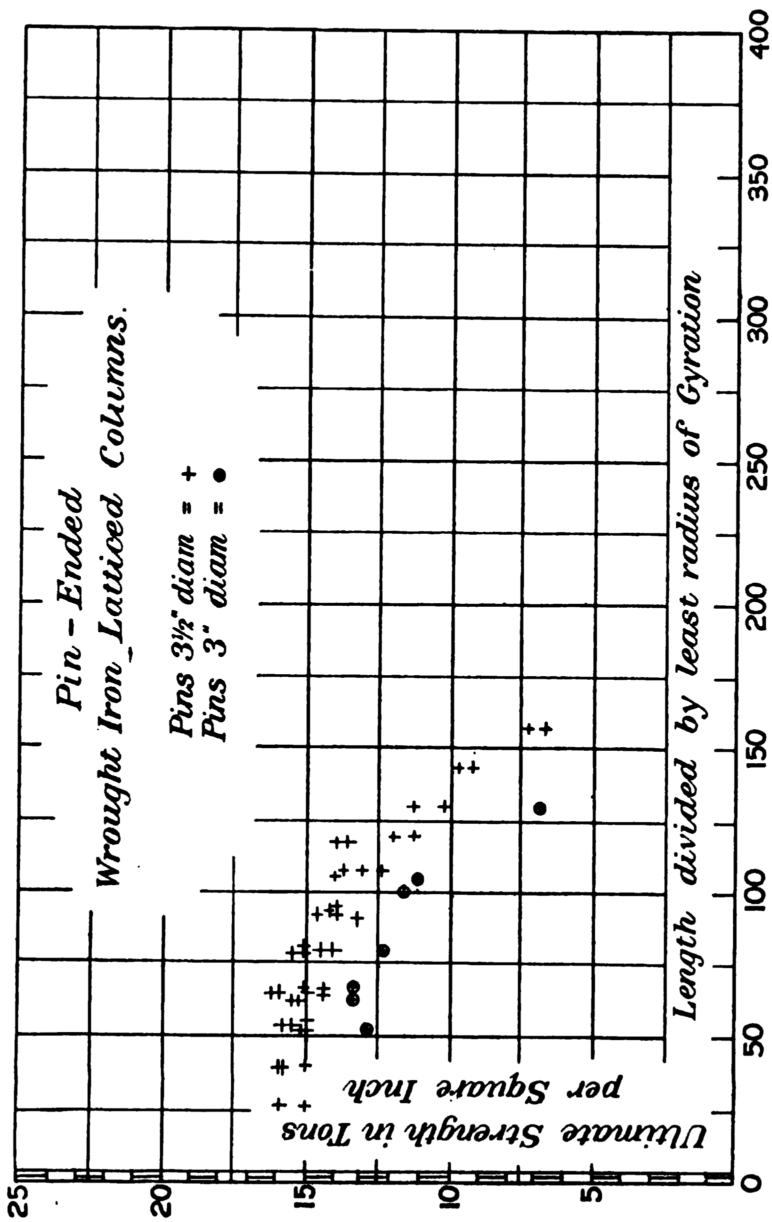


Fig. 133.

destruction. One column was flat-ended at one end and pin-ended at the other, with a total length of 15 feet from centre

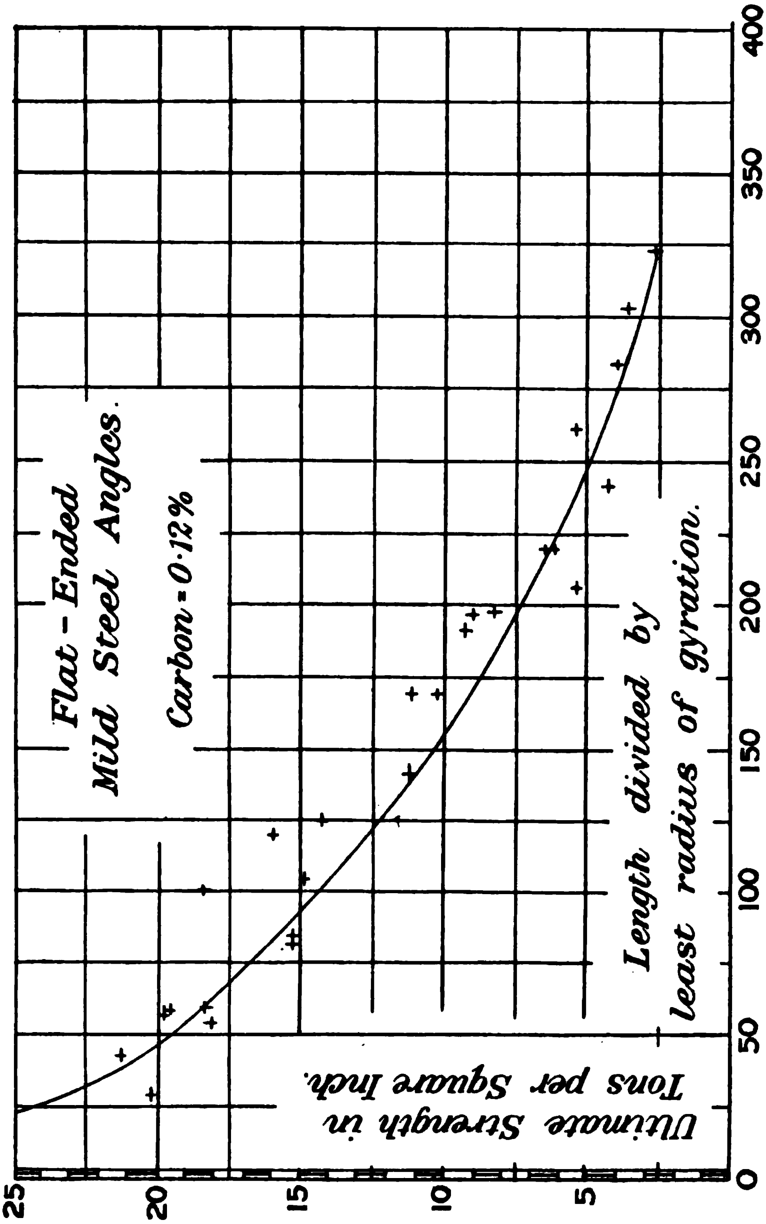


FIG. 184.

of pin to flat end of column. The section consisted of one flange plate 11″ × $\frac{5}{16}$″, two side plates 10″ × $\frac{1}{4}$″, and four angles

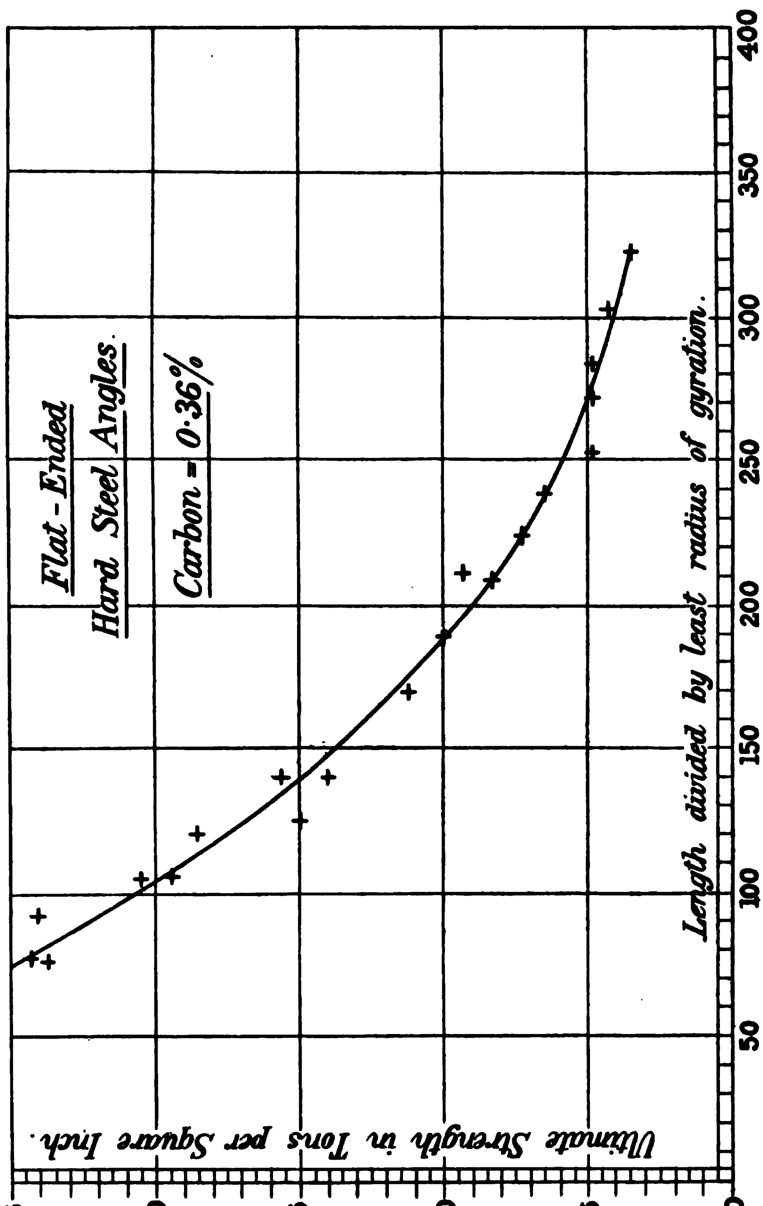


FIG. 135.

$2\frac{1}{4}'' \times 2\frac{1}{4}'' \times \frac{1}{4}''$. This column failed by flexure and local buckling at 20·9 tons per square inch.

The second column was of similar type, but of larger scantlings, flat-ended at both ends, 24 feet $1\frac{1}{2}$ inch in length, and consisted of one flange plate $13\frac{1}{2}'' \times \frac{5}{16}''$, two side plates $12'' \times \frac{1}{4}''$, two angles $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$, and two angles $2\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$.

This column also failed by flexure or local buckling at 19·3 tons per square inch. The average ultimate tensile strength of the angles and plates was 35·3 tons per square inch, with 22 per cent. extension in 8 inches. The steel, therefore, was of somewhat harder quality than that generally referred to in Chapter I.

From the representation of the practical results obtained from the testing machine, we may now pass to the consideration of the breaking weights of columns as proposed by various authorities who have approached the subject either from a theoretical standpoint, or who have proposed formulæ more or less empirical to embody the ascertained results of experiment.

In Figs. 136 and 137 are plotted the values given by Professor T. Claxton Fidler for fixed-ended and round-ended columns of hard steel, mild steel, wrought iron, and cast iron. For the details of the mathematical treatment of the subject, the student is referred to the original works of that author.[1]

In Fig. 138 are plotted the values for the breaking weights of wrought-iron columns, both flat-ended and pin-ended, of various sections, as given in the formulæ proposed by an American authority. Similarly the values of the breaking weights of mild steel, wrought-iron, and cast-iron columns, flat- and pin- or round-ended, as given by the formulæ proposed by another American authority, are plotted in Fig. 139.

We may now proceed to the discussion of various practical sections of struts and columns of rolled mild steel, such as are usually found in ordinary construction.

Figs. 140 to 175 give typical sections of struts and columns commencing from the simplest and most elementary forms, and exhibiting the growth or evolution of the more complex sections produced by riveted combinations of simple forms. It is to be remembered that the figures are type sections only. The proportions of the various members and their relative sectional areas and positions will be determined in every case by the conditions to be

[1] Professor T. Claxton Fidler, "A Practical Treatise on Bridge Construction." See also *Proc. Inst. Civil Engineers*, vol. lxxxvi.

met, and the amount of load to be carried, whether considered as a direct simple vertical load, or a combination, as is often the case,

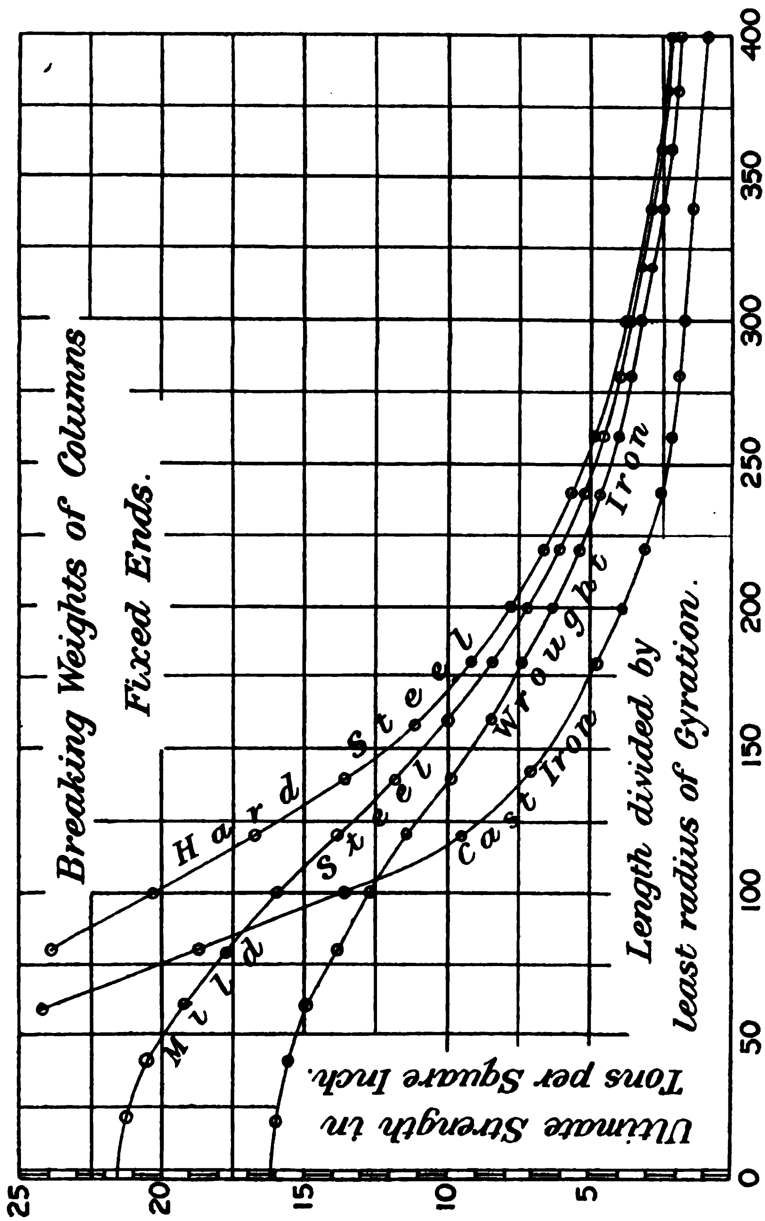


FIG. 186.

of vertical and transverse loading. The designer will endeavour naturally in cases of direct vertical load to equalize as far as

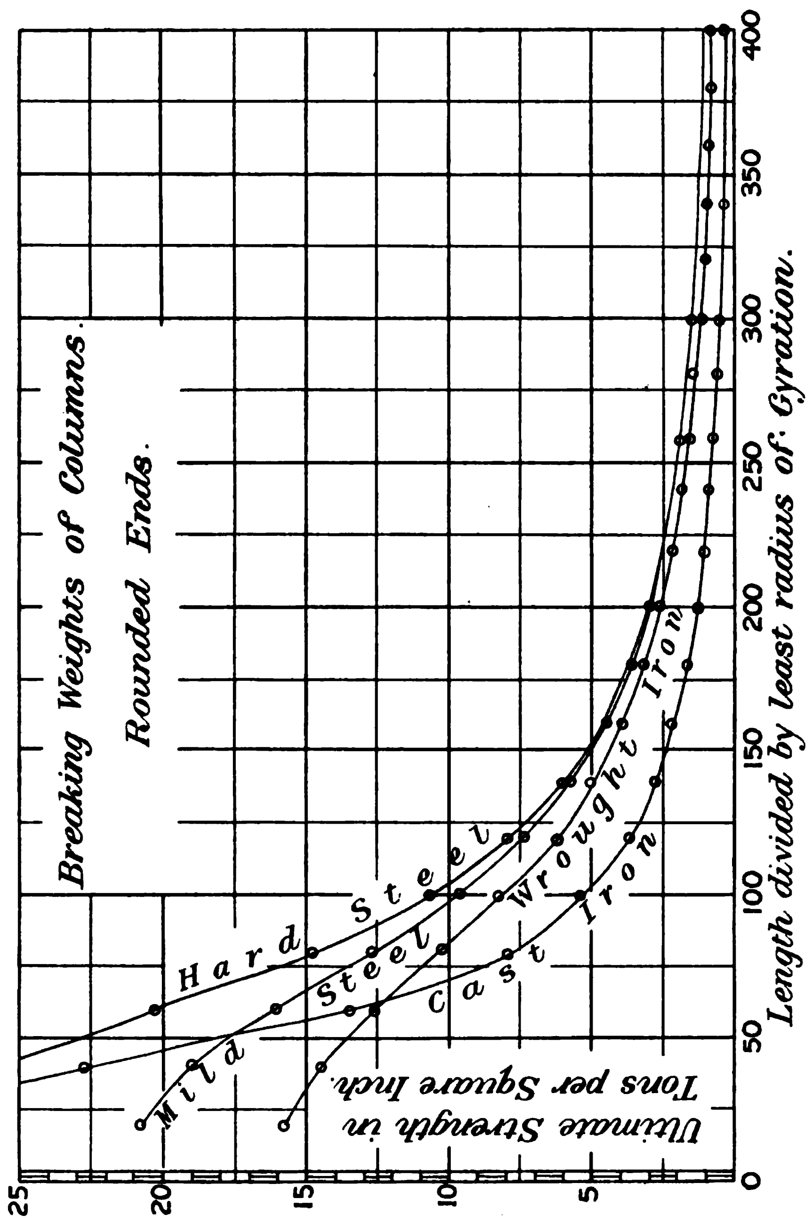


FIG. 187.

possible the radii of gyration about all axes, while disposing his metal at the greatest possible distance from the neutral axis; but it

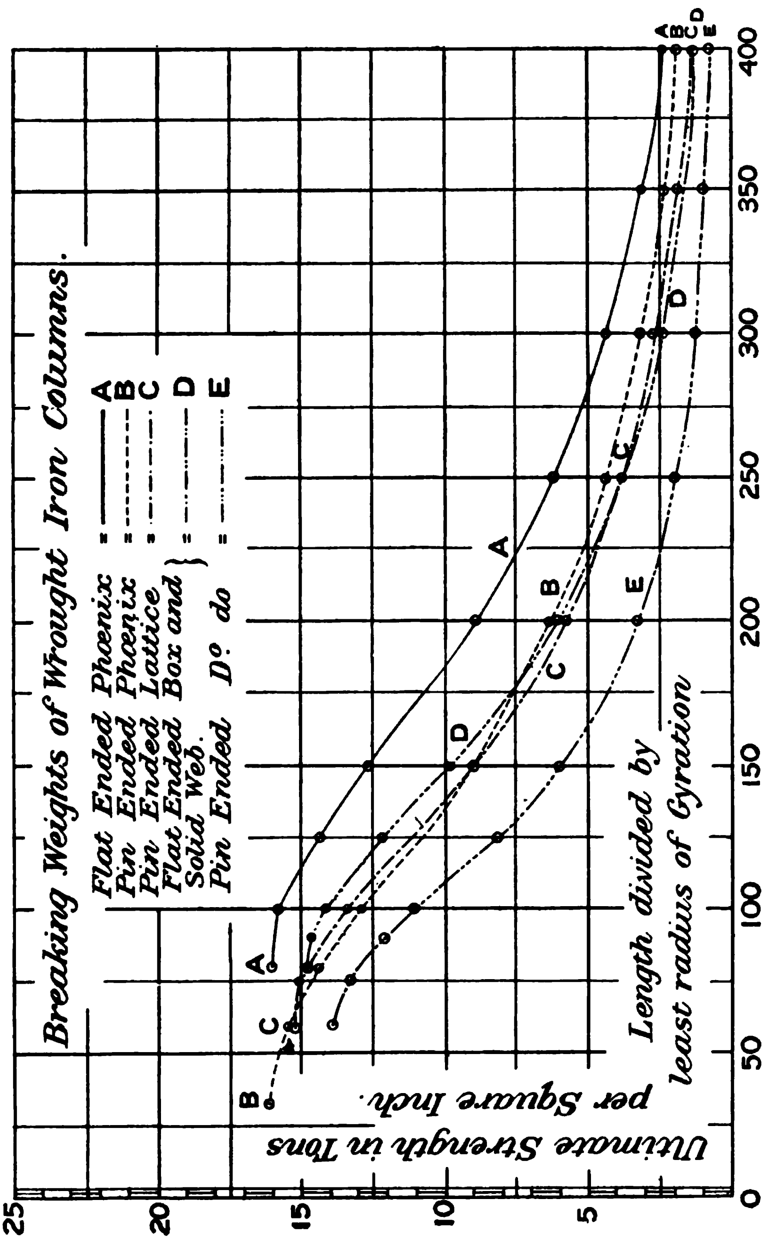


FIG. 138.

will frequently happen that this is not feasible, and, in fact, is only theoretically met by the adoption of perfectly circular sections,

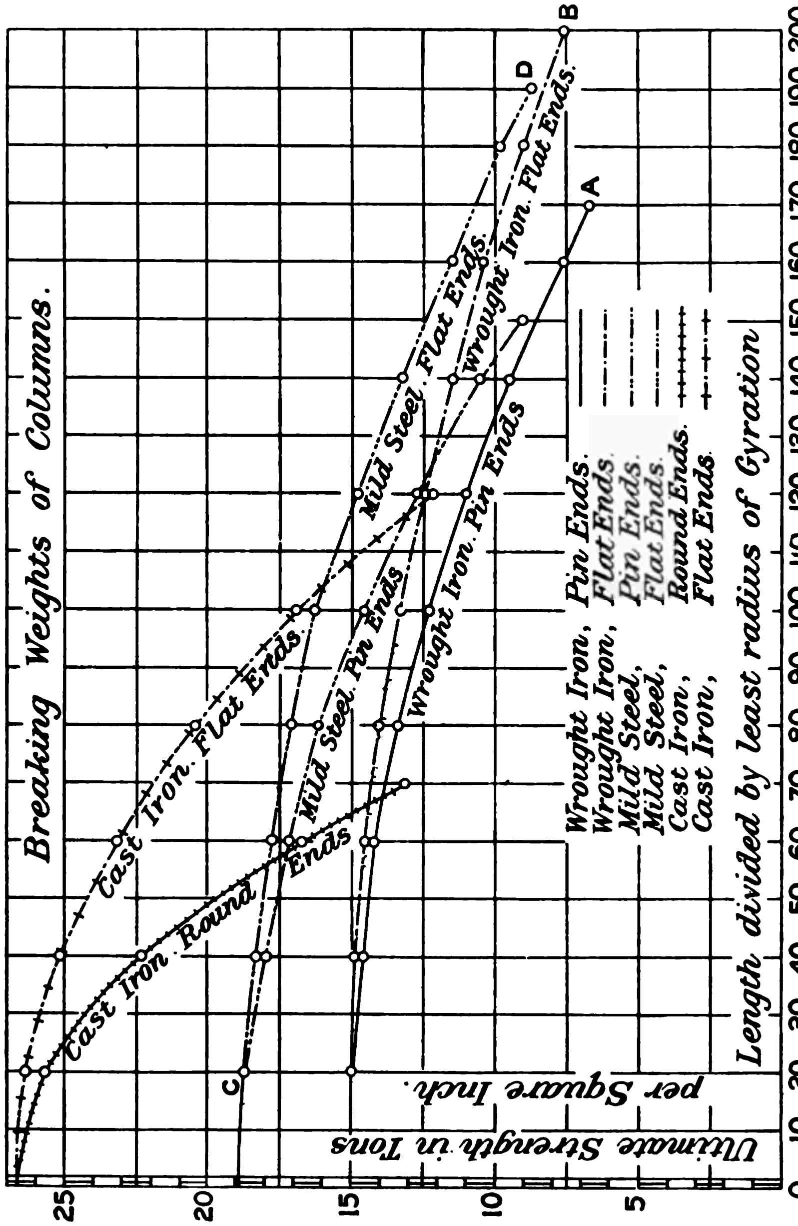


FIG. 139.

such as the hollow circular cast-iron column or welded tube. In all cases of unequal moments of inertia about different axes and unequal radii of gyration, the proportions of $\frac{l}{r}$ must be determined with reference to the least radius of gyration or the direction in which failure by flexure may be expected.

Fig. 140, a simple flat bar, is met with as a strut or compression member in the webs of multiple lattice girders. It is obviously weak in the direction of its least radius of gyration, and is prevented from failure in that direction by the frequent intersection of the tension diagonals and the use of stiffening vertical members. Outside the particular application mentioned, it is not frequently used as a compression member, being obviously less adapted for that purpose than other and stiffer sections.

Fig. 141 is an example of the use of flat bars in pairs, frequently used as compression members in the diagonals of roof principals of

FIG. 140.

FIG. 141.

FIG. 142.

FIG. 143.

small span, say up to 30 or 40 feet or thereabouts. The bars are usually pin-connected at the ends, and swelled apart at the centre by cast-iron distance pieces as shown. It is a convenient form of strut for light loads, but is apt to fail by weakness at the ends, the bars buckling near the pin connections.

Fig. 142 is a simple angle, either equal or unequal legged section. This section is in common use in the compression diagonals of small lattice girders and as a single angle in the rafters of small and light roof principals and their diagonals.

The double angle shown in Fig. 143 is a variation of the same type used for the same purposes, where the loads are heavier. The angles are occasionally riveted close together, back to back, and thus form practically a riveted tee. The mode of attachment of the single or double angle as a compression member is nearly always by one leg only. Under these conditions the distribution of stress is conceivably very unequal over the whole cross-section, and it is to be regretted that while numerous experiments have

been carried out upon the ultimate strength of angles in compression, so little has been done to elucidate under actual practical working conditions the ultimate strength of angles connected in the usual way, and with the direction of the compressing forces out of centre with the centre of figure of the section.

A further elaboration of the use of angles is given in Fig. 144, which is a somewhat special section, occasionally used in the compression diagonals of roof trusses of large span. The four angles

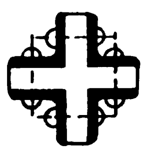


FIG. 144.

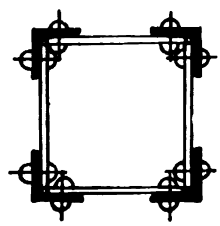


FIG. 145.

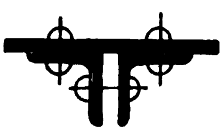


FIG. 146.

are brought near together at the ends and swelled out at the centre by cast-iron distance pieces.

A more simple example of the use of four angles is given in Fig. 145, also used as a compression member of large roof trusses, and consisting of four angles connected by internal stiffening plates at intervals as shown, or by light lattice bracing in the

FIG. 147.

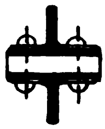


FIG. 148.

FIG. 149.

same planes. Such a strut is frequently pin ended, but may also be riveted and "fixed" ended. This strut is an example on a small scale of the type shown in Fig. 164, which is intended for large columns with heavy loading.

Fig. 146 will be at once recognized as the compression or top flange of ordinary plate or lattice girder construction of moderate spans. The increase of sectional area required is usually obtained by increasing the thickness or number of the plates.

Fig. 147, showing a simple tee section, is in common use as the rafter or compression member of roof trusses up to about 40-feet span, and in their compression diagonals. Its place is sometimes taken by the double angles shown in Fig. 143. It is

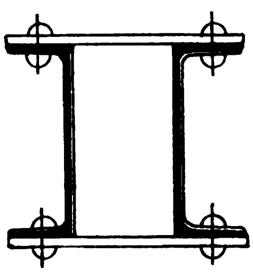


FIG. 150.

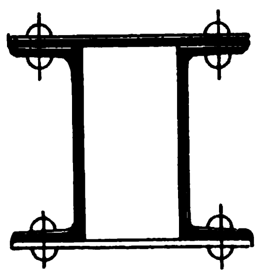


FIG. 151.

also frequently used as the upper flange of small lattice girders or trussed purlins in roof work.

The double tee (Fig. 148) is used in the compression diagonals of large roof trusses, especially in those types of "crescent"-shaped principals shown in Fig. 345, where the stresses in the diagonals are not great. In such cases the double tees are brought near

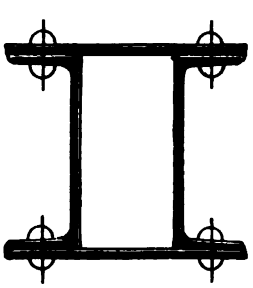


FIG. 152.

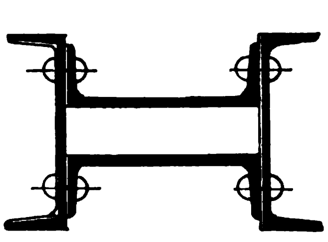


FIG. 153.

together at the ends and swelled out in the centre with cast-iron distance pieces, and are not so liable to the local weaknesses that may occur with the use of double flats, as in Fig. 141.

Fig. 149 is the simple channel, frequently used alone, but perhaps more commonly in combination, either in pairs, as in Figs. 150, 152, or as in Fig. 161.

Fig. 150 gives a pair of channels connected by lattice bars on both sides, and forming an open section used in the compression rafters of roofs of considerable span, say from 70 to 100 feet, as ordinary columns, or in compression members in large triangular girders.

Fig. 151 is the same combination of channels, but with a solid

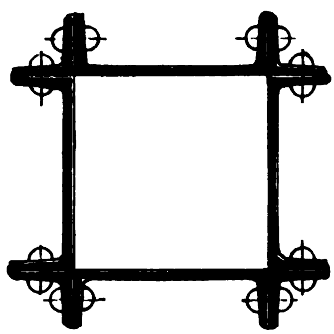


FIG. 154.

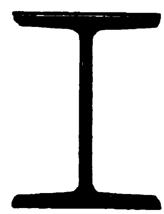


FIG. 155.

plate connection on the one side and open latticing on the other, used for similar purposes to those above mentioned, but where heavier stresses have to be provided for.

Fig. 152 shows a section of column frequently used for heavy

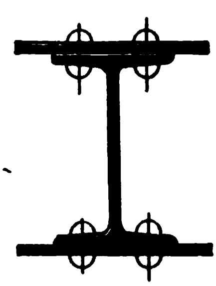


FIG. 156.

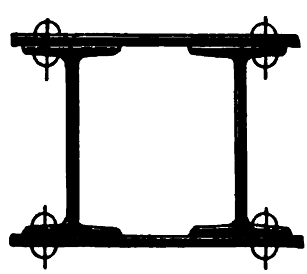


FIG. 157.

loads in buildings, warehouses, dock sheds, and the like, consisting of two channels and two solid plate sides forming a closed cell.

Fig. 153 is a form of section composed of four channels as shown.

Fig. 154 shows an effective section consisting of four channels connected at the external corners by four angles.

We now approach a group of sections in which the rolled joist is the principal feature, used to a very large extent in ordinary building construction, and which combine a considerable amount of stiffness with simplicity and ease of construction, and economy in riveting.

Fig. 155 is the simple rolled joist, in which no riveting is required except in end connections when used as a plain column or strut. The principal defect in this section is the inequality of the radii of gyration round the axes severally square to the web and flanges. The relative values of these radii will be found for various sections in the table of the mechanical elements of rolled joists (p. 97), and in columns or struts exposed to lateral shock the liability to flexure in a plane square to the web must be borne in mind. This defect has apparently been recognized by some

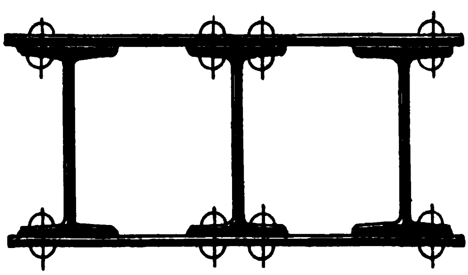


FIG. 158.

manufacturers, who have produced a section of rolled joist of exceptional width in the flange. This weakness is also to some extent corrected in the next development of this form of column shown in Fig. 156, where plates (one or more in thickness) are riveted to the flanges of the rolled joist, whereby the moment of inertia round the axis parallel to the web is increased. This section is very useful, and is largely used in columns for general building purposes.

Fig. 157 shows a pair of rolled joists, connected by plates as shown, or by a system of lattice bars in the same planes. A practical example of this type on a large scale will be referred to in detail hereafter.

Fig. 158 shows a strong column for heavy loads, composed of three rolled joists connected by plates as shown, or by latticing.

Fig. 159 shows a combination of three rolled joists which is the

prototype in miniature of the more elaborate section shown in Fig. 167.

Fig. 160 shows the same combination, with the addition of

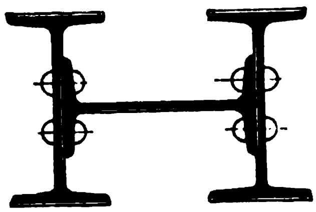


Fig. 159.

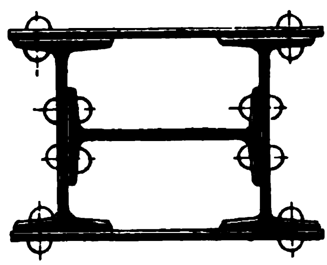


Fig. 160.

external flange plates, which may be replaced either by latticing or flat stiffening plates at intervals.

Fig. 161 gives a column composed of one rolled joist and two channels, a simpler form of the type shown in Fig. 165.

It not unfrequently happens that a built-up section of plates and angles, though more expensive, offers greater facilities to the

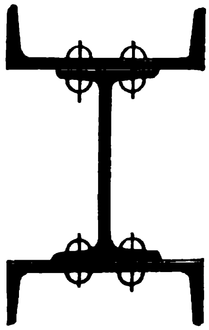


Fig. 161.

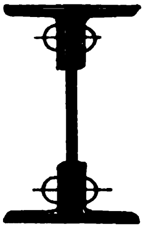


Fig. 162.

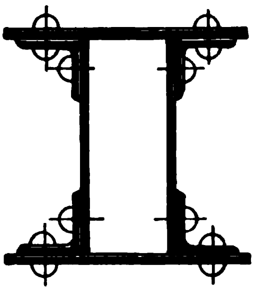


Fig. 163.

designer in certain details of connections or in arrangement of cross-section than a simple rolled section of similar type of outline, and so we frequently find the built-up section consisting of a plate and four angles, shown in Fig. 162. This section, similarly to that shown in Fig. 156, may be further elaborated by the addition of flange plates riveted to the angles.

Fig. 163 gives a box section of great strength and stiffness,

frequently used in columns carrying heavy loads. The same amount of metal disposed as shown in Fig. 164 will yield a greater uniformity in the value of the radii of gyration about different

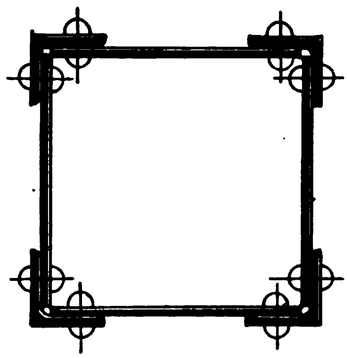


FIG. 164.

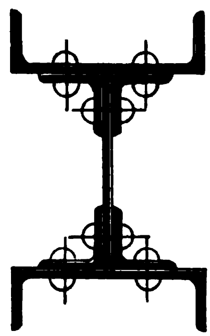


FIG. 165.

axes, but the riveting is more difficult to get at, and must be dealt with in a manner similar to that adopted in ships' masts, sheer legs, derricks, etc.

Fig. 165 gives a valuable section, of good appearance, and

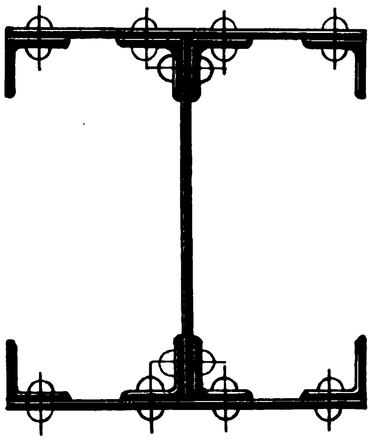


FIG. 166.

great stiffness in all planes, composed by the addition of channels riveted to the section shown in Fig. 162.

Figs. 166 and 167 show types of built-up-sections of plates and

angles adapted to meet special conditions in large columns carrying heavy and diverse loadings.

The use of these sections will be further alluded to in detail.

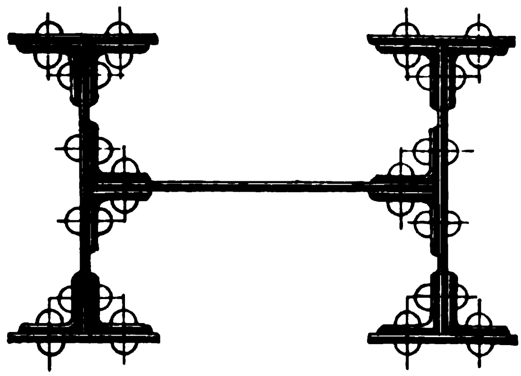


FIG. 167.

Figs. 168 to 170 give sections of columns composed largely of Zed sections combined with plates or latticing.

Fig. 168 is composed of four Zeds and one central plate. Fig.

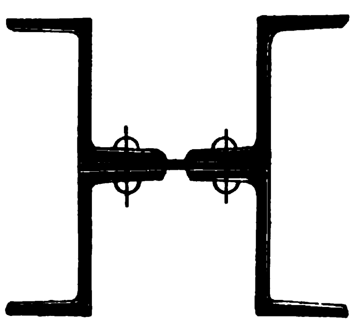


FIG. 168.

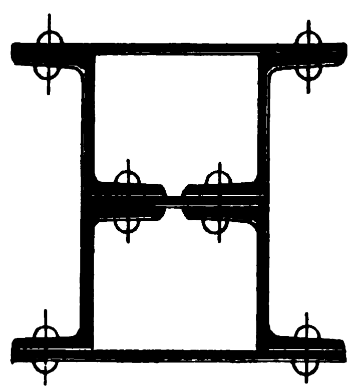


FIG. 169.

169 is of similar section, with additional plates on the outside. Latticing may take the place of these outside plates.

Fig. 170 shows a similar combination with the Zeds turned the reverse way, the metal being disposed to better advantage, though the appearance of the column is perhaps not so satisfactory.

Figs. 171, 172 are sections of a more or less special nature,

somewhat less simple in their end connections and the details connecting them with other members than the types above considered.

Fig. 171 is composed of four tees, or four sets of double angles,

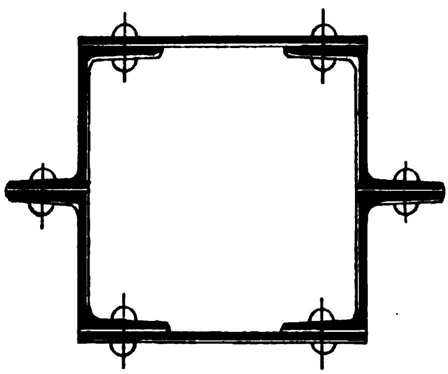


FIG. 170.

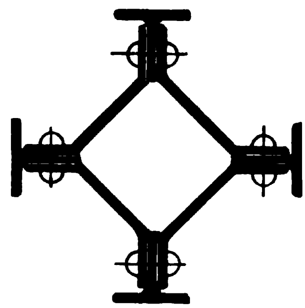


FIG. 171.

disposed as shown and connected by bent plates, or trough sections.

Fig. 172 is a section of a type of column made up of the so-called "trough sections" used for flooring and decking. This

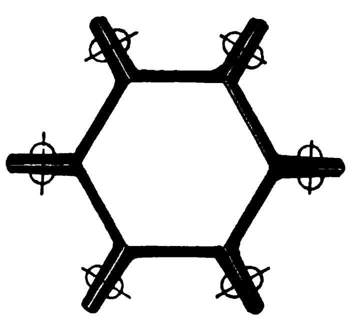


FIG. 172.

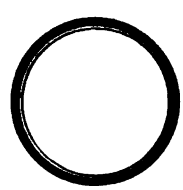


FIG. 173.

combination produces a column of great strength and stiffness, though not so well adapted for secondary connections as others.

The use of the circular column in any other material than cast

iron in ordinary building construction is somewhat limited. Its use in cast iron in the type of section shown in Fig. 173 is too universal and well known to need any further description.

The circular section in mild steel may in small columns take the form of welded tubes, and in larger sections of plates bent to a circular curve and butt-jointed with covers.

In this form we find its use on a large scale in sheer legs, ships' masts, derrick poles, and occasionally in bridge work of very large span. In such structures the circular plate is frequently stiffened internally, and in sections of sufficient size manual labour inside the tube is used for the purposes of riveting.

In Figs. 174 and 175 we have the section generally known as the Phœnix. The figure gives an arrangement in four sections only, but a larger number may be employed in accordance with

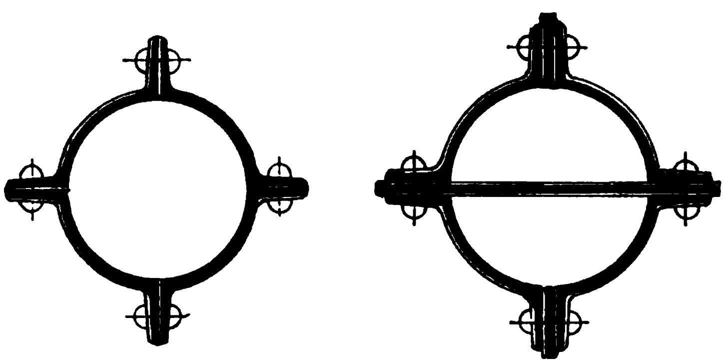


FIG. 174. FIG. 175.

the size required. This form is one of great stiffness, and in wrought iron has given very high results in the testing machine, possibly owing to the mutual support given by the form of section and the stiffening ribs, and its consequent freedom from local weakness in the unsupported part of the plate.

Secondary attachments offer some little difficulty with this form of section, and the type most frequently found is that shown in Fig. 175, where filling strips are inserted between the flanges of the segments. This gives the opportunity of insertion of sketch plates wherever required for attachments, the filling strips being stopped off as required. Occasionally the strips are carried right through the diameter of the column.

If we now endeavour to institute a comparison between the

sections which have been above described, based either on the grounds of efficiency or economy in first cost, it must be premised that any section can hardly be considered *per se* without reference to its surroundings, and the use to which it has to be put.

The requirements in detail of the various secondary members which may have to be attached to the simple column will always have an influence in the choice of selection. The design in detail of the cap and base, the attachments for such fittings as counter-shafting brackets, the counterbracing as in the case of the piers to a viaduct, the attachments of traveller, roof, or floor girders will invariably demand careful consideration, and the success of the design as a whole will be influenced by the skill with which these details are worked out.

As regards economy in first cost, other things being equal, it may be assumed that the section having the least amount of riveting will be the cheapest per unit of weight. Thus comparing the built-up sections, we find that Figs. 162, 168 show two lines of rivets, Fig. 155, which is the equivalent of Fig. 162, having no rivets. Figs. 152, 156, 157, 159, 161 will have four lines of rivets, Figs. 169, 170 have six lines, Figs. 158, 160, 163, 164 have eight lines, Fig. 166 has ten lines, while Fig. 167 has eighteen lines.

The extent, however, to which the use of simple sections, with a comparatively small amount of riveting, can be carried, is governed by all the conditions of the case, and the amount of load to be carried.

A further comparison may be made of the sections above described which is not without importance, and that is the extent to which the surfaces of the respective sections can be protected from the effects of corrosion, or, in other words, the extent to which the sections can be got at by the paint brush.

All the simple sections are fairly accessible, Figs. 151, 157, 160, having latticing on one or both sides, can be painted internally, while Figs. 163, 164, 169, 170, with solid plate flanges, have closed cells, which cannot easily be painted under ordinary conditions. Such closed cells are not infrequently filled with concrete, although the extent to which this acts as a preservative coating depends largely on the degree of close contact with the metal obtainable.

Figs. 155, 156, 159, 161, 162, and 165 to 168 can be readily painted inside and out. Figs. 152, 157, 158, 160, 163, 169, 170

will be either closed cells or accessible to the paint brush, if they have solid plate flanges or latticing respectively.

A temptation, often present to the mind of the designer when preparing his working drawing for a column or strut, prompts him to save time and trouble by only detailing, say, the cap and base to some convenient scale, and breaking off the remainder so as not to show to a true scale the entire height and length of the column or strut. This is a practice not to be commended, and one which the junior draughtsman is cautioned against, inasmuch as the opportunity is lost of viewing the true proportions of diameter or least dimension to height, a factor which is always of great importance.

The complete elevation in true scale enables the trained eye of an experienced designer to verify the theoretical conclusions he may have arrived at as to the proportions of his column, and may save him the embarrassment of the after-contemplation of a work which may either look dangerously slender or unnecessarily stout.

The above remark is perhaps the more necessary, inasmuch as in the following details of columns which are now presented, owing to the exigencies of illustration, the details of the column will be given rather than the complete elevation.

The examples which follow are taken from the working drawings of columns actually constructed.

Fig. 176 is the side elevation of a riveted steel column of the type shown in Fig. 162, for supporting a heavy warehouse floor with roof over, the building consisting of ground and first floor only.

The section consists of four angles, four flange plates, and one solid web. In order to distribute the heavy concentrated load properly over the concrete bed on which it stands, a cast-iron base plate is employed, shown in side elevation in Fig. 177, in end elevation in Fig. 178, and in plan in Fig. 176.

The meeting surfaces of the base plate and column would in such a case be machined carefully square to the vertical axis of the column, and the bed-plate fixed carefully level, and the concrete surface grouted up, or prepared with sheet lead or felt.

In Figs. 181 and 182 we have shown the upper portion of a similar type of column of somewhat lighter section, showing the details of the cap.

The connection of the cap of the column shown in Fig. 177, with

the floor girders and the base of the column above, is an illustration of one method of dealing with a question which will constantly

FIG. 176.
Scale $\frac{1}{3}$ inch = 1 foot.

FIG. 177.
Scale $\frac{1}{2}$ inch = 1 foot.

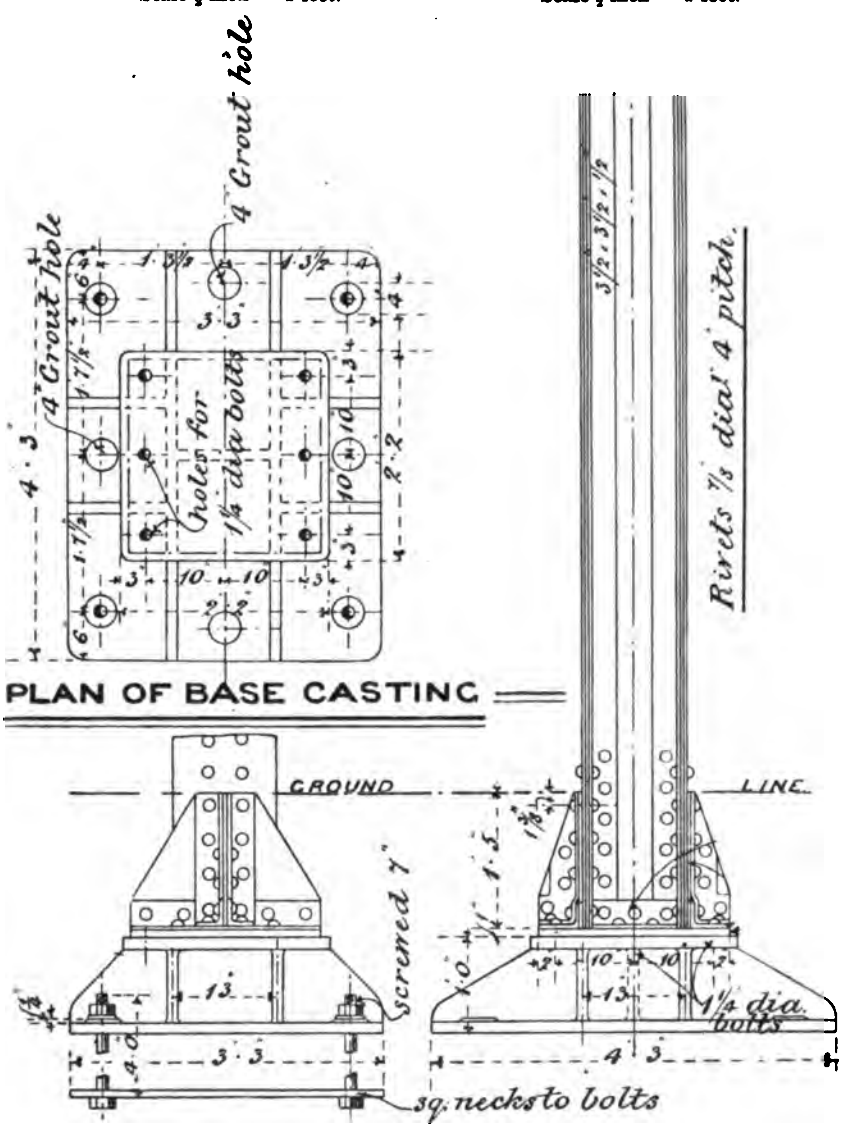
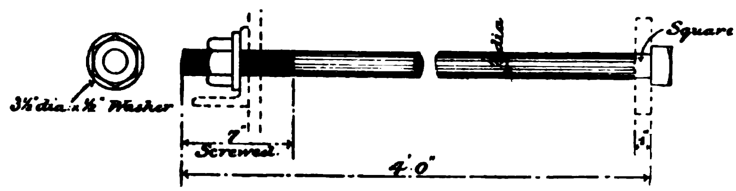


FIG. 178.
Scale $\frac{1}{2}$ inch = 1 foot.

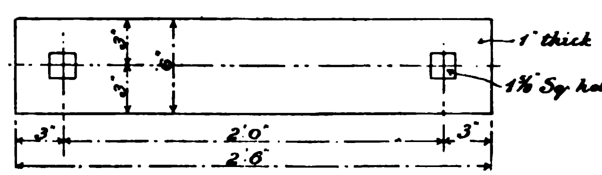
present itself to the mind of the designer, viz. whether the column should give way to the connections at each floor, or *vice versâ*, or, in other words, whether the columns should be in one continuous length from top to bottom, or broken wherever attachments for floor girders have to be made. In the writer's view no hard-and-fast rule can be laid down. Where the column spacing is wide,



HOLDING-DOWN BOLTS.
(4 THUS TO EACH COLUMN)
FIG. 179.
Scale 1 inch = 1 foot.

and the floor girders, as in the case under consideration, are of considerable span, carrying heavy loads, it appeared desirable that the detail of girder seating should occupy the first place, especially when the column next above, carrying the roof load only, is of moderate scantling and the load light.

In pursuance of the above reasoning, Figs. 183 and 184 show the front and side elevations of the cap of the column arranged to



WASHER PLATES.
(2 THUS TO EACH COLUMN.)
FIG. 180.
Scale 1 inch = 1 foot.

take the ends of the principal and secondary floor girders, and it will be observed that the base of the upper column, which, as above stated, carries the roof load only, stands, not immediately upon the cap of the lower column, but upon the upper flanges of the floor girders, the compressive stresses being brought down to the lower column by means of the webs of the girders, which are shown in

detail in sectional plan in Fig. 185, and which are strengthened by additional plates and angles to enable them to do this duty. The base of the column above is shown in sectional plan in Fig. 186, on the line HH (Figs. 183 and 184).

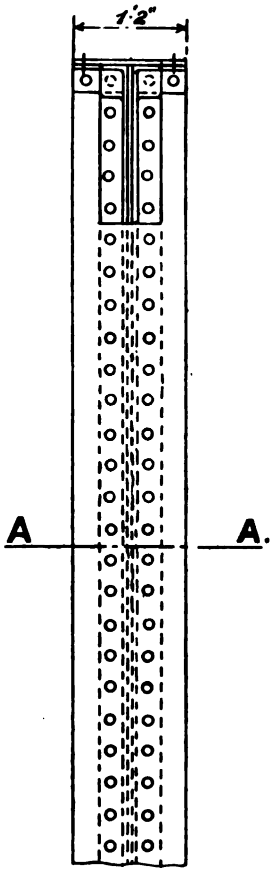


FIG. 181.
Scale ¼ inch = 1 foot.

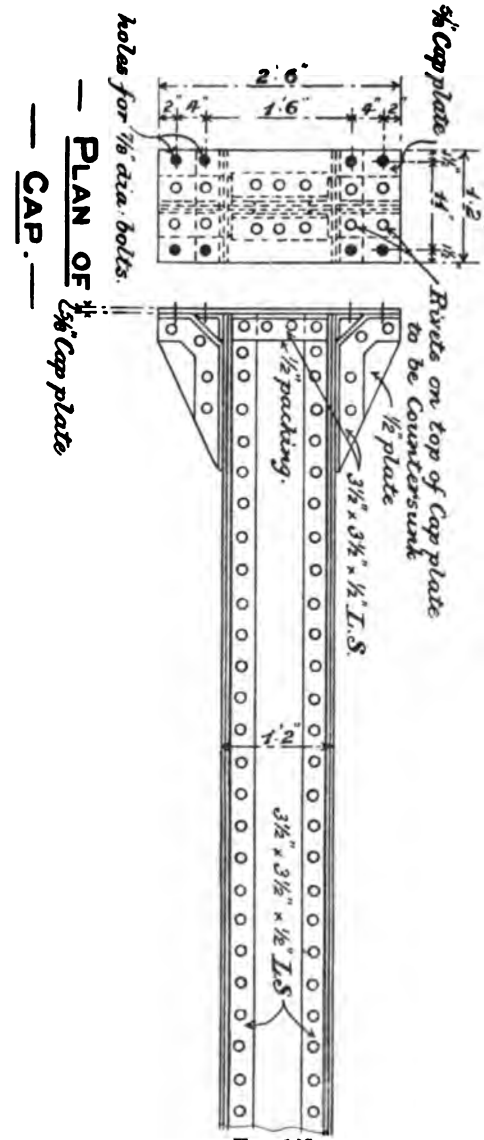


FIG. 182.
Scale ¼ inch = 1 foot.

These figures then illustrate a case where the continuity of the column structure has been broken in order to meet girder requirements. But it must not be supposed that this is an example to be followed where the conditions are not similar.

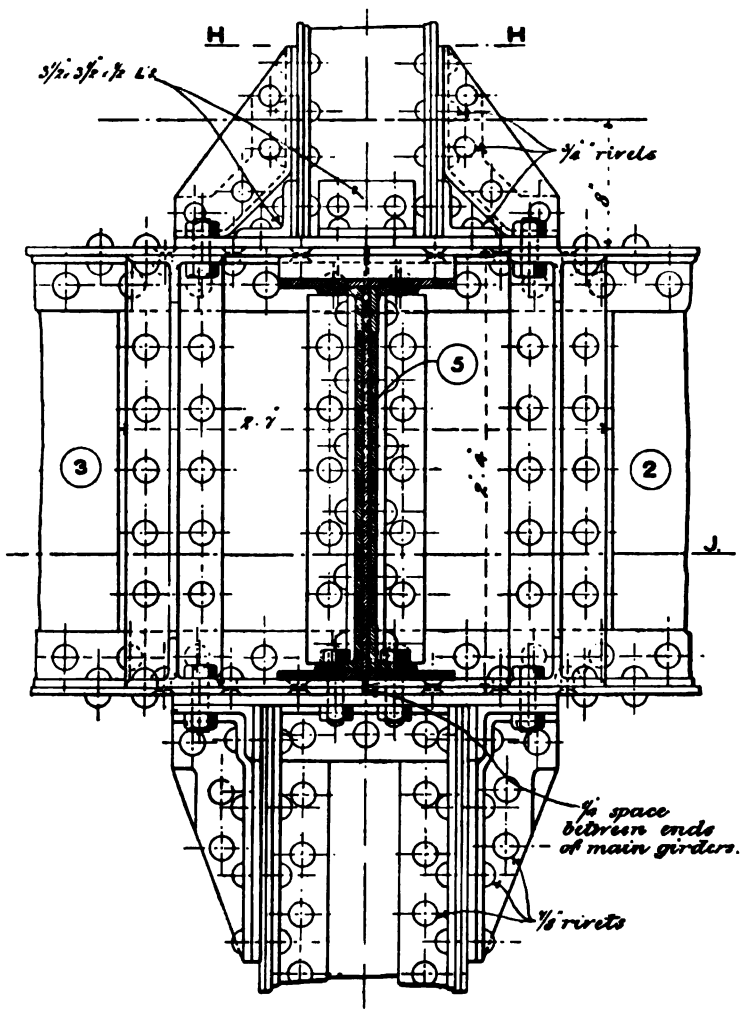


FIG. 183.
Scale 1 inch = 1 foot.

In cases of buildings of several stories, where the column is considerable height, in segments of the various floor spacings,

and where the accumulation of load on the lower columns is heavy, a sound judgment would lead to the conclusion that the continuity of the column is of the first importance, and that all

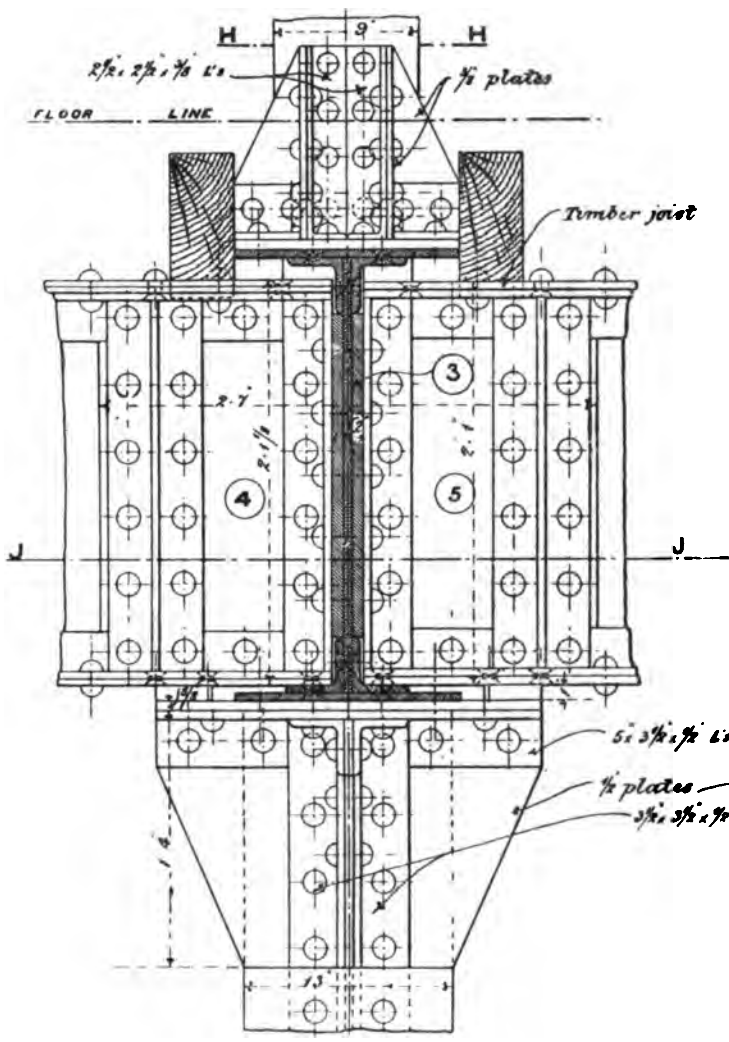
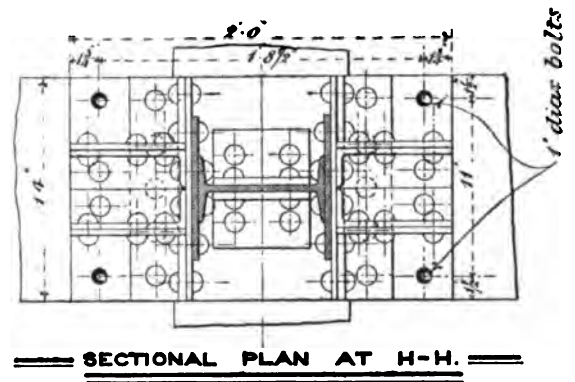


Fig. 184.
Scale 1 inch = 1 foot.

details of girder connections should be made to give way to it. Such a case might be conceived to arise in the columns supporting

the various box and gallery tiers in a theatre auditorium, and where the safety of the audience demands that the utmost care be

FIG. 186.
Scale 1 inch = 1 foot.



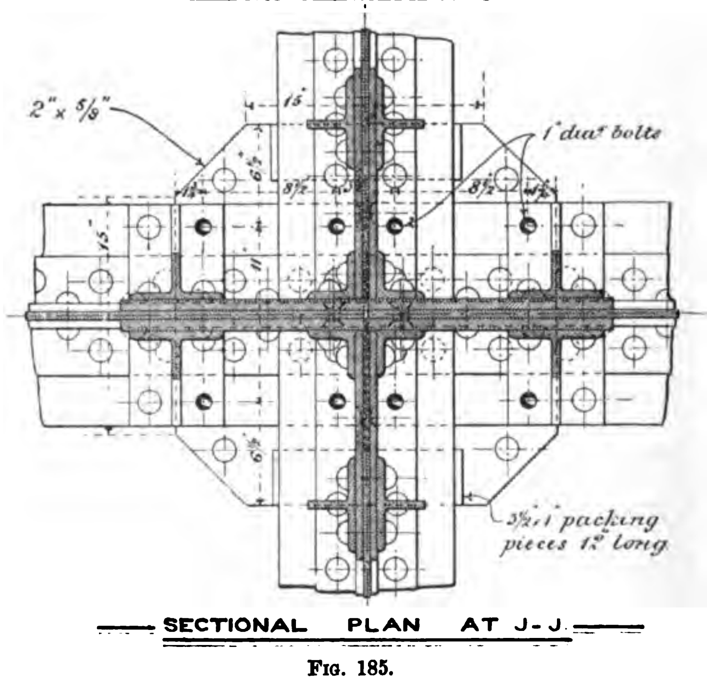


FIG. 185.
Scale 1 inch = 1 foot.

taken in the design of all column details. Even in this case the continuity of the column has not in practice always been preserved,

although by the use of divided girders carrying the box tiers and forming the projecting cantilever portions, this desirable end could easily be secured.

Another illustration of the importance of column continuity is found in the very lofty structures erected on the principle known as skeleton steel construction, as commonly practised in the United States. Here the consideration of transverse stresses due to wind pressure, and the necessity of preventing lateral flexure or oscillation, leads to very careful consideration of the column connections, and the best practice demands as much continuity of the column members as can be practically attained, the girder seatings being as a rule bracketed off the columns.

We may have, then, two leading ideas which point to the desirability of column continuity, the accumulation of vertical loading, and the possibility of lateral flexure or oscillation in the building as a whole.

Figs. 187, 188 give details of the base of a column of similar type to the foregoing, *i.e.* of girder section, of the type shown in Fig. 162. In this case no cast-iron bed-plate is used, the base of the column being arranged to bear directly upon concrete or stonework.

We may next examine the details of a massive column of the type shown in Fig. 166, used in the interior of an engine-house, and supporting a heavy load arising from two lines of traveller girders, together with the load of girders and cast-iron tank above, the details of which are considered on pages 158 to 171.

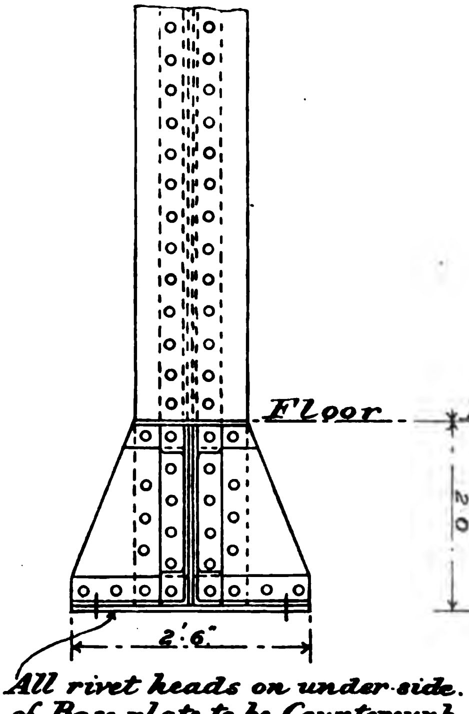
It will be seen from Figs. 189 and 190 that at the level of the capping or bracketing supporting the traveller girder, the column is gathered in, and reduced in section to the dimensions shown on Fig. 191, which is a plan of the top of the traveller girder seatings looking down. The upper column is of the same type of section as the lower portion, and possesses great stiffness in all directions. Fig. 189 is a front section on the line E of Fig. 191, and Fig. 190 is a section on the line DD of Fig. 191.

It will be observed that the traveller girder seatings are borne, as it were, upon the shoulders of the lower portion of the column, while the continuity and rigidity of the column as a whole is well maintained.

The clearance between centre of traveller rail and face of column is 9 inches, less the projection of the rivet heads (see the remarks on this detail in Chap. III., p. 137), a clearance which is sufficient for the type of traveller used in this instance.

The normal section of the lower portion of the column below traveller girder seatings is shown in Fig. 192, and of the upper

Fig. 187.
Scale ½ inch = 1 foot.



Note.—The holding-down bolts to this column are four in number, 1¼" diameter, with flat bar washer plates 1" thick, and 3¼" diameter washers, as shown in detail in Figs. 179, 180, p. 226.

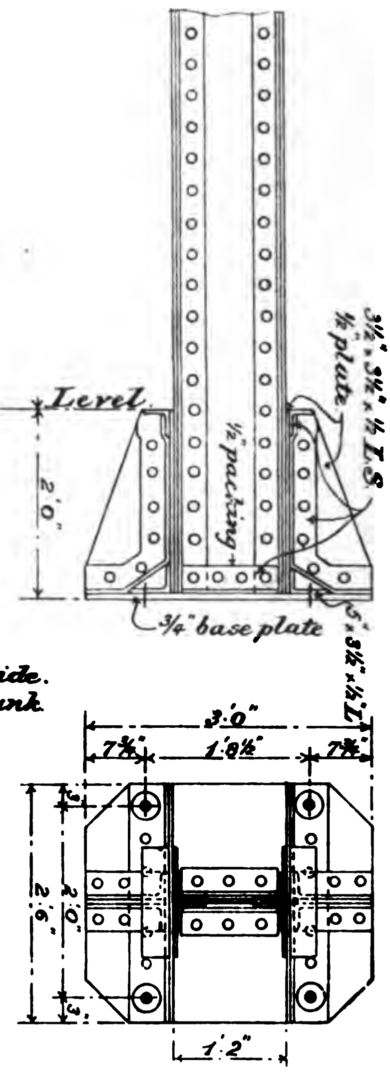


Fig. 188.
Scale ½ inch = 1 foot.

portion in Fig. 194, the side elevation and plan of cap to upper portion of column being shown in Figs. 195, 196.

In columns of this type, where the height is considerable, it is

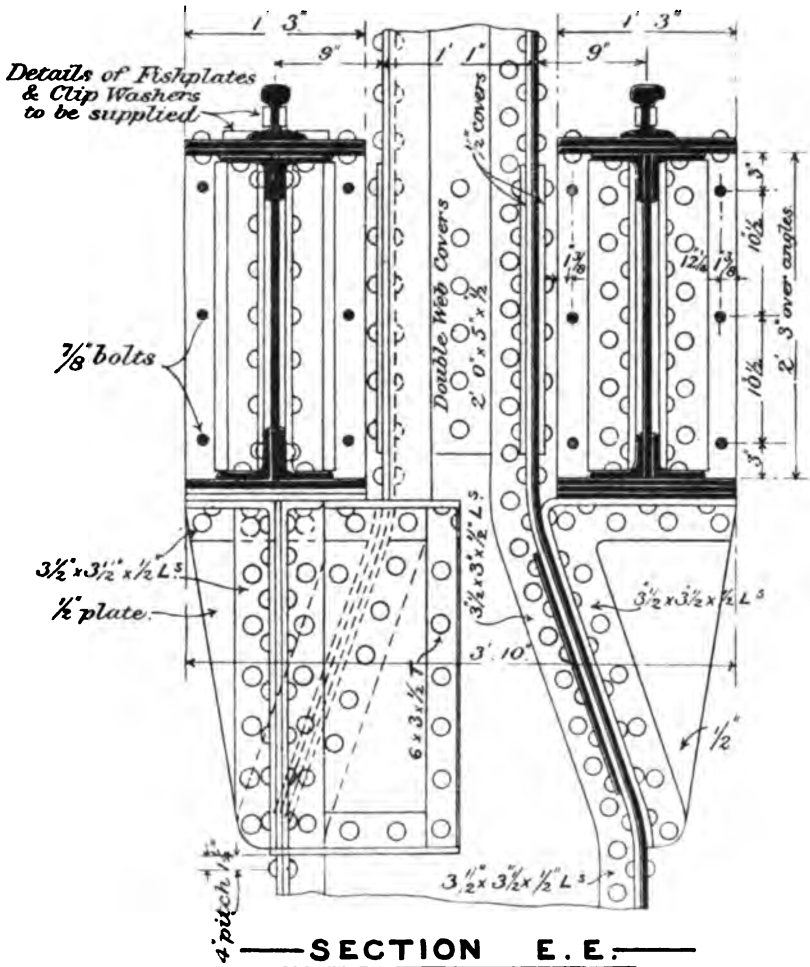


Fig. 189.
Scale ⅜ inch = 1 foot.

sometimes desirable to introduce plate and angle stiffeners, similar to the stiffeners in the web of an ordinary plate girder, for the purpose of preventing distortion or flexure in the outer corner angle irons. This detail is shown in Figs. 192, 193.

A column performing similar functions to the one last described, and of similar type, is shown in Figs. 197 to 205.

The method of effecting the connection between the upper and

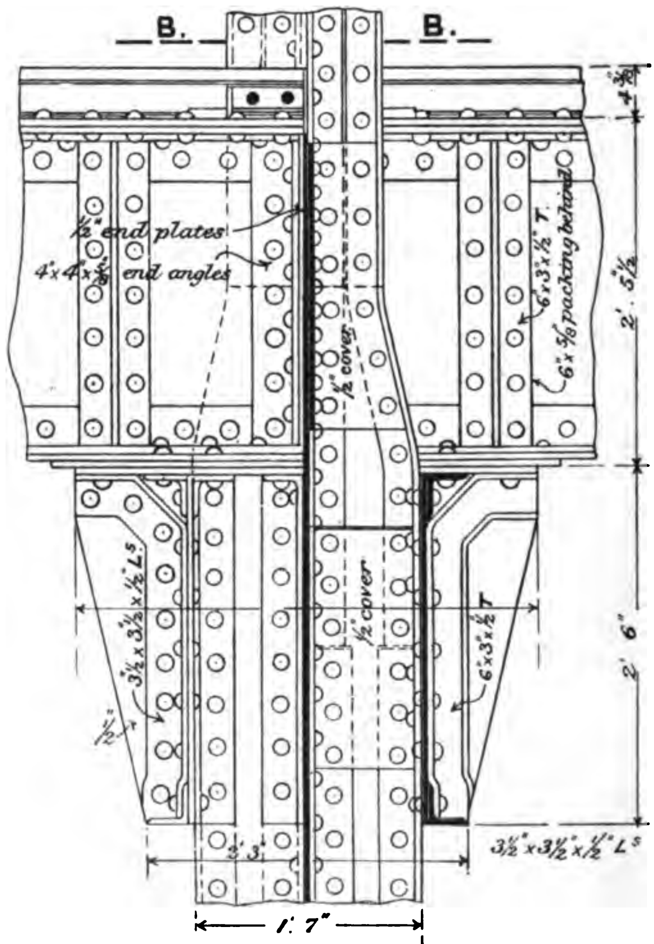
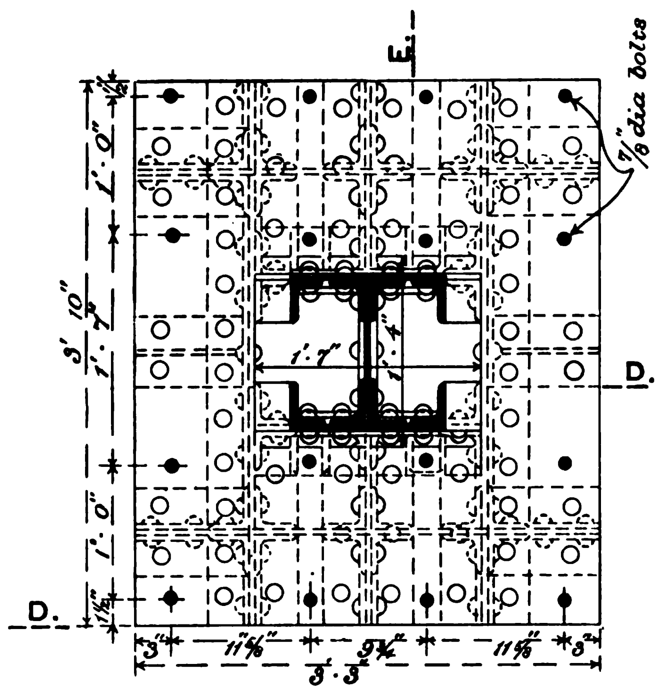


FIG. 190.
Scale ⅜ inch = 1 foot.

lower sections of the column at the level of the traveller girder is shown in Figs. 197 and 198, which is a variation from the design shown in Figs. 189 and 190. The normal section of the lower

portion of the column is shown in Fig. 200, together with the details of the stiffener shown in plan in Fig. 200, and in elevation

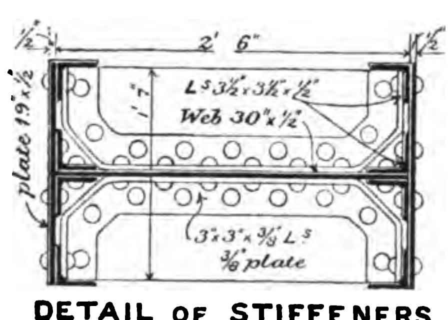


PLAN AT B.B.

GIRDERS OMITTED

Fig. 191.

Scale ¾ inch = 1 foot.



DETAIL OF STIFFENERS.

IN LOWER COLUMN.

Fig. 192.

Scale ¾ inch = 1 foot.

in Fig. 201. Details of the base of this column are shown in Figs. 202, 203, 204, and 205, with lewis bolts securing the column to a masonry bedstone.

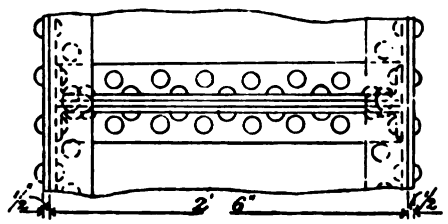


Fig. 193.
Scale ¾ inch = 1 foot.

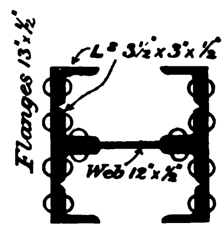


Fig. 194.
Scale ¾ inch = 1 foot.

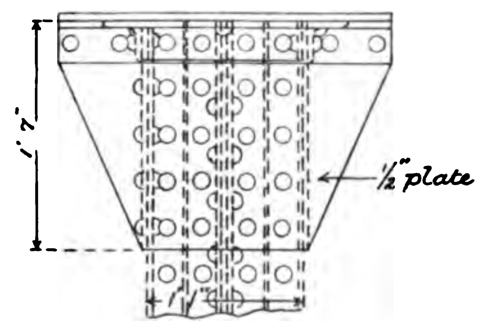
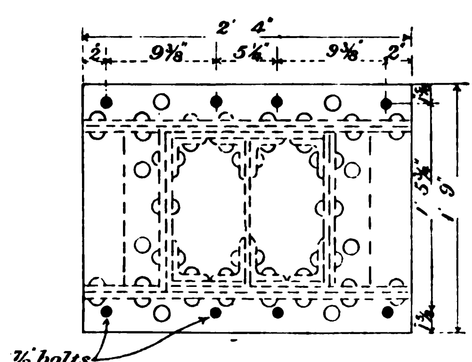


Fig. 195.
Scale ¾ inch = 1 foot.



PLAN OF CAP.

Fig. 196.
Scale ¾ inch = 1 foot.

A column, of which the simple elementary type is shown in Fig. 157, is shown in detail in Figs. 206 to 215 inclusive.

This column consists, in its lower portion, of two rolled joists

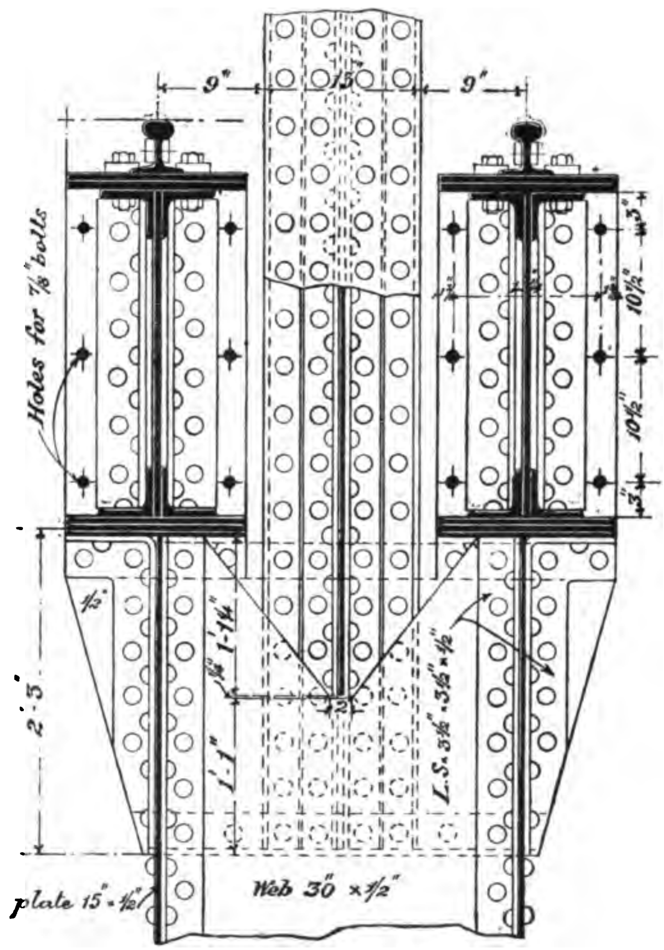


FIG. 197.
Scale ½ inch = 1 foot.

strengthened with flange plates, and connected together into one member by the parallel system of flat bar lattices, shown in elevation in Figs. 206 and 207, which are side elevations of the column

at the base and cap respectively. The front elevation is given in Figs. 208 and 209, also at the base and cap respectively. The normal section of the lower portion is given in Fig. 210. It will be seen from the figure that the flat bar braces are slightly bent to pass over one another at the intersection, where they are

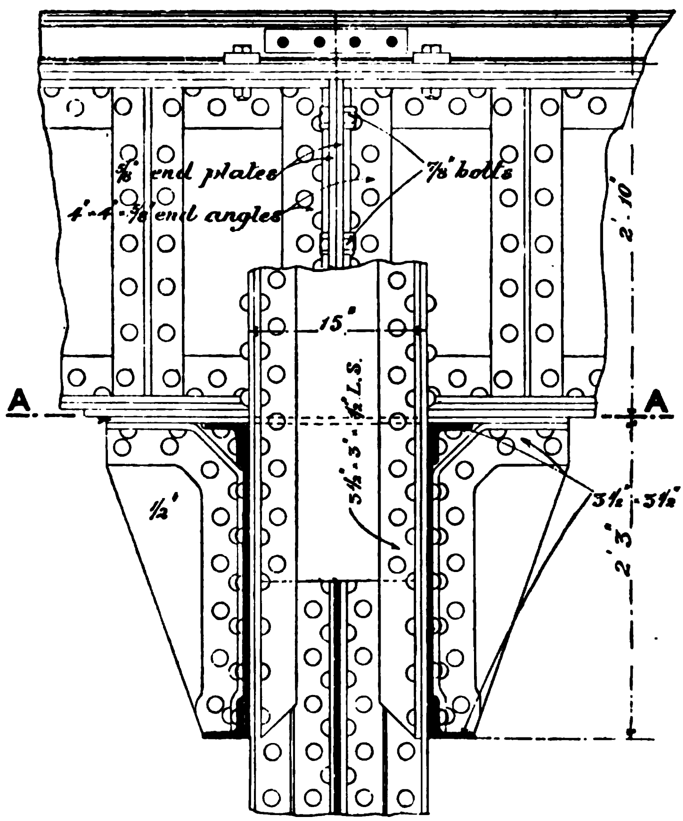


Fig. 198.
Scale ⅜ inch = 1 foot.

riveted together, while the flat bar bracing, consisting of 4" × ⅝" flats riveted together at the meeting face, is intended to prevent any tendency to a corkscrew twist in the rolled joist members of the column. The weight-bearing section is also seen to consist of two rolled joists 16 inches deep at 62 lbs., each with two 8" × ½" plates riveted on.

A sectional elevation of the base is shown in Fig. 211, on the line DD of Fig. 212.

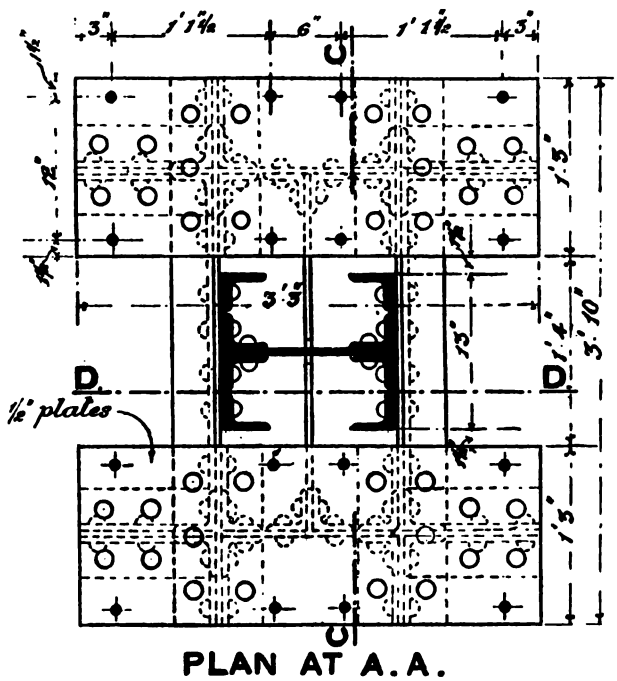


FIG. 199.
Scale ⅜ inch = 1 foot.

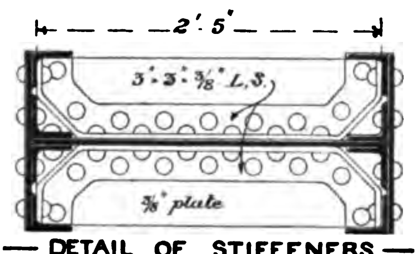


FIG. 200.
Scale ⅜ inch = 1 foot.

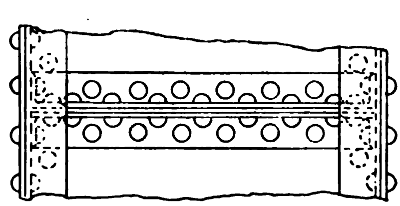


FIG. 201.
Scale ⅜ inch = 1 foot.

A sectional plan of the column on the line CC of Fig. 206, looking down, is given in Fig. 212, from which and Fig. 206 it will

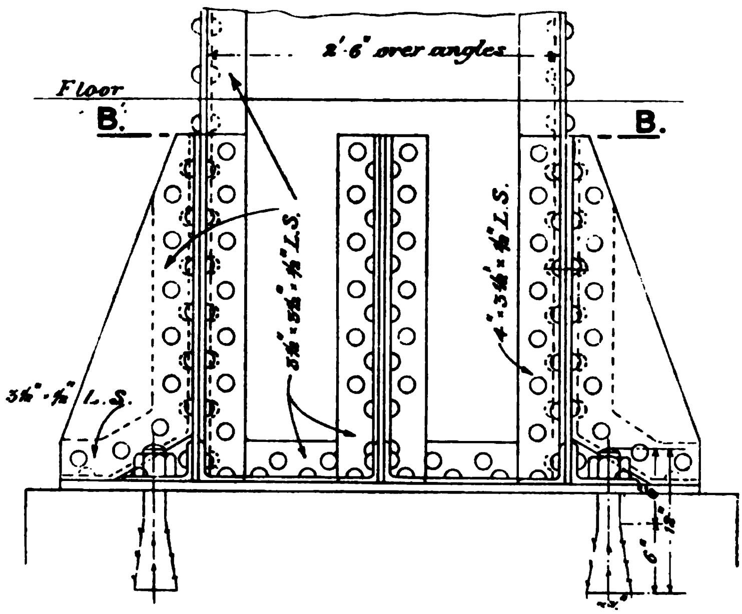
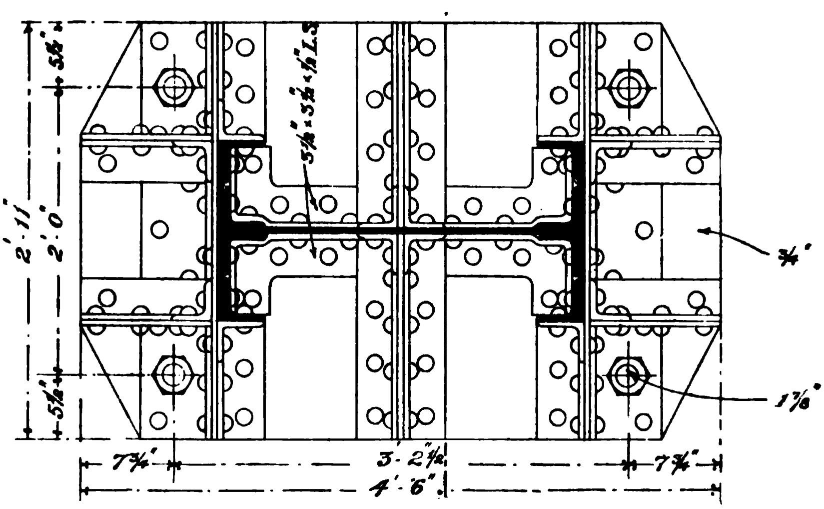


Fig. 202.
Scale ¾ inch = 1 foot.



PLAN AT B. B.

Fig. 203.
Scale ¾ inch = 1 foot.

be seen that the column is held down by four holding-down bolts, which in this case pass through a bed of concrete.

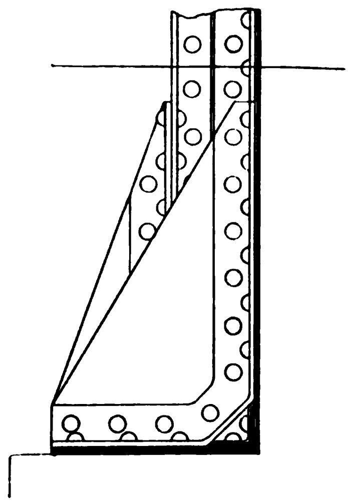


FIG. 204.
Scale ¾ inch – 1 foot.

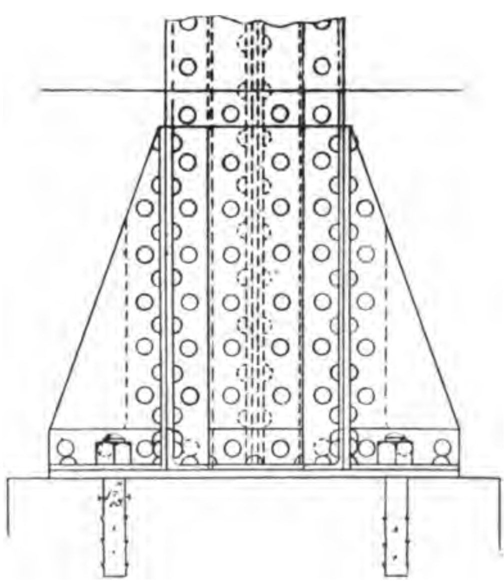


FIG. 205
Scale ¾ inch = 1 foot.

The column carries two lines of traveller girders, while the upper section, continued above the traveller girder seatings, carries

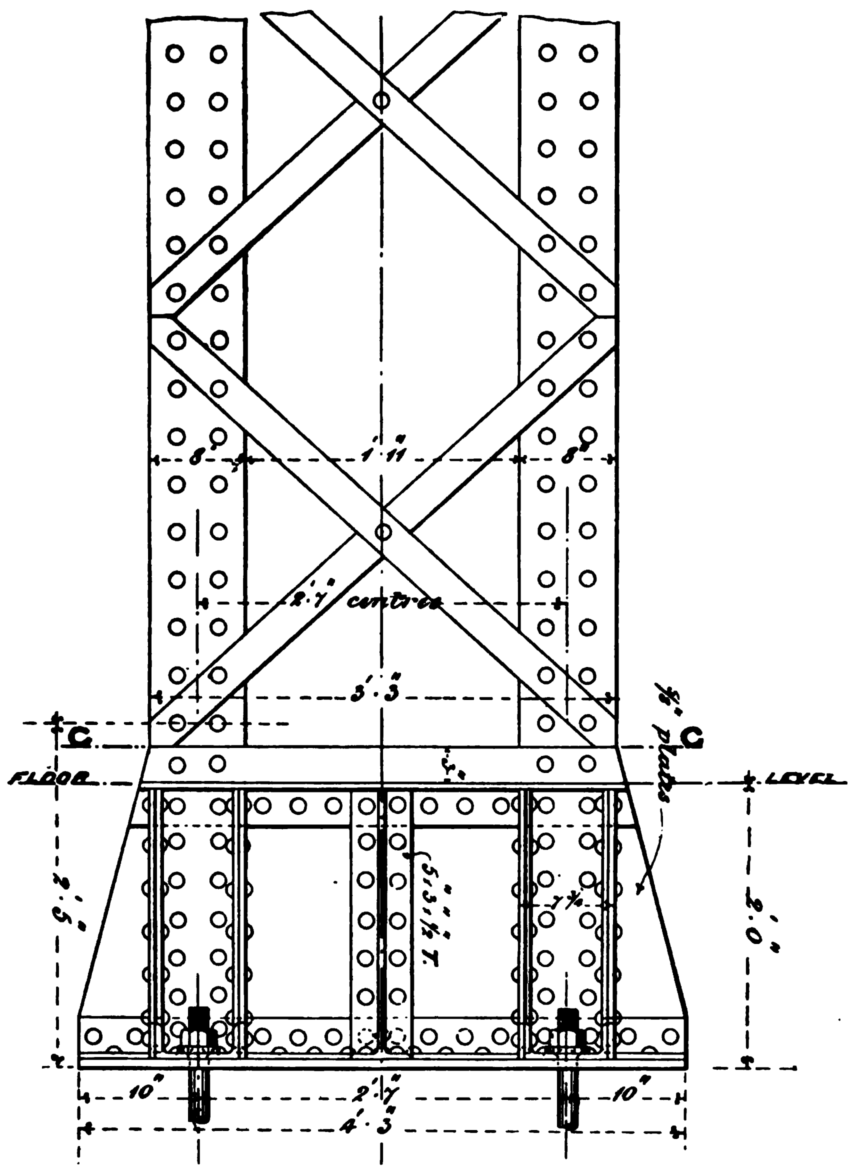


Fig. 206.
Scale ¾ inch = 1 foot.

a roof load. The connection between the upper and lower sections of the column, at the level of the traveller girders, is shown in

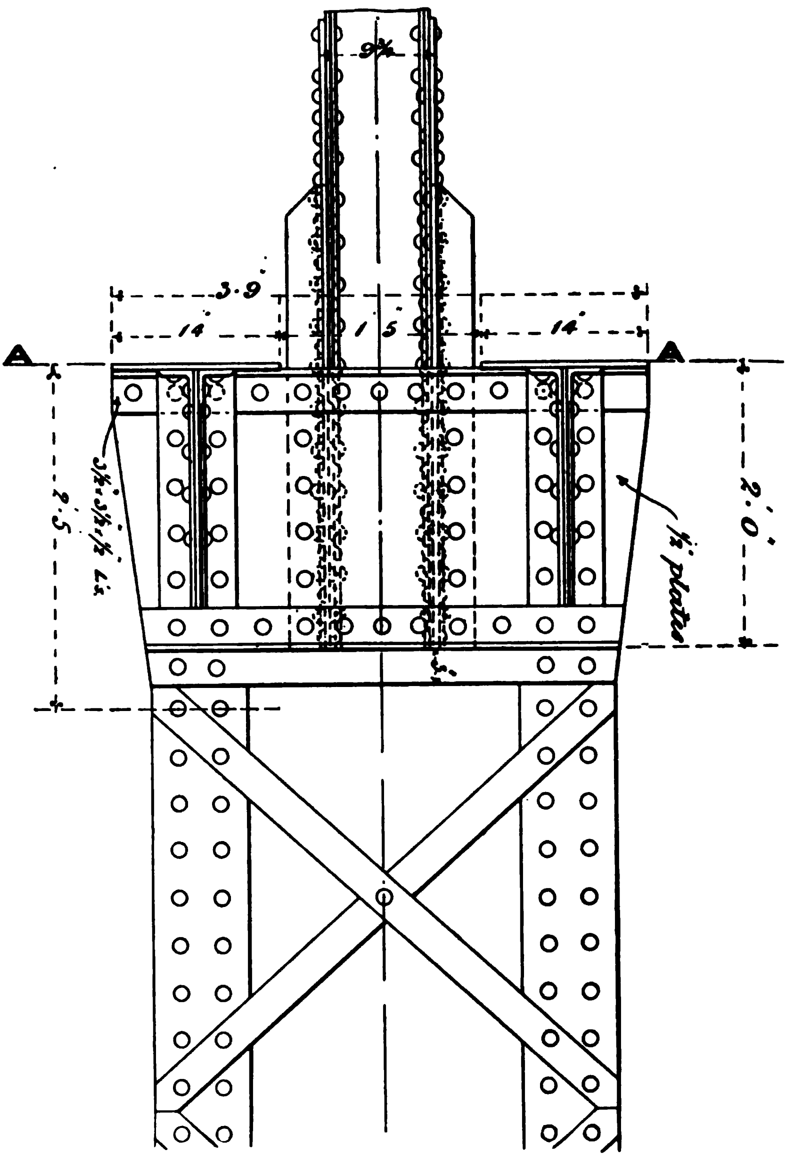


FIG. 207.
Scale ¾ inch = 1 foot.

Figs. 207, 209, 213, and 214. Fig. 213 is a sectional plan, looking down, on the line AA of Fig. 207, and shows the upper faces of the traveller girder seatings.

Fig. 214 is a vertical section on the line FF of Fig. 213. It will be seen that the normal section of the upper portion of the

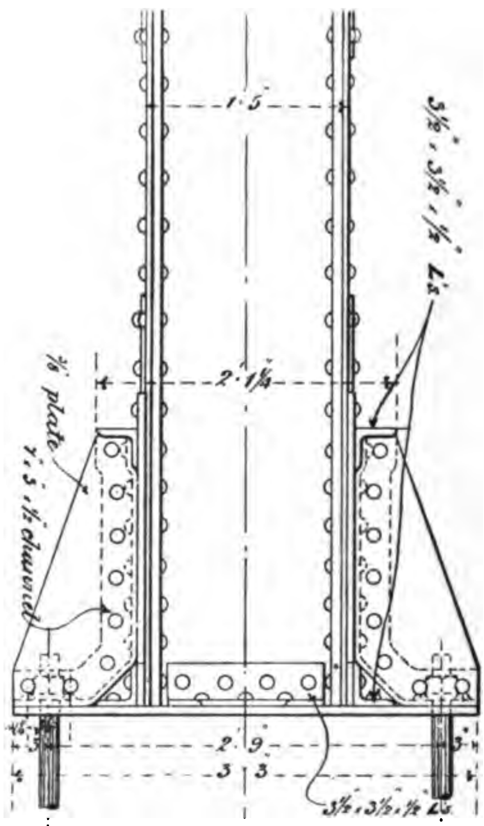


FIG. 208.
Scale ¾ inch = 1 foot.

column is of the type shown in Fig. 156, and a plan of its cap is shown in Fig. 215.

It will be observed that the load arising from the upper section is distributed equally over the two members of the lower section by means of the plates and angles forming the cap and

seatings of the traveller girders. This member has to act, therefore, as a girder, being subject to transverse and shearing stresses, while the loads arising from the traveller girders are transmitted directly down the two principal members of the lower column.

A column of somewhat complex type will now be described,

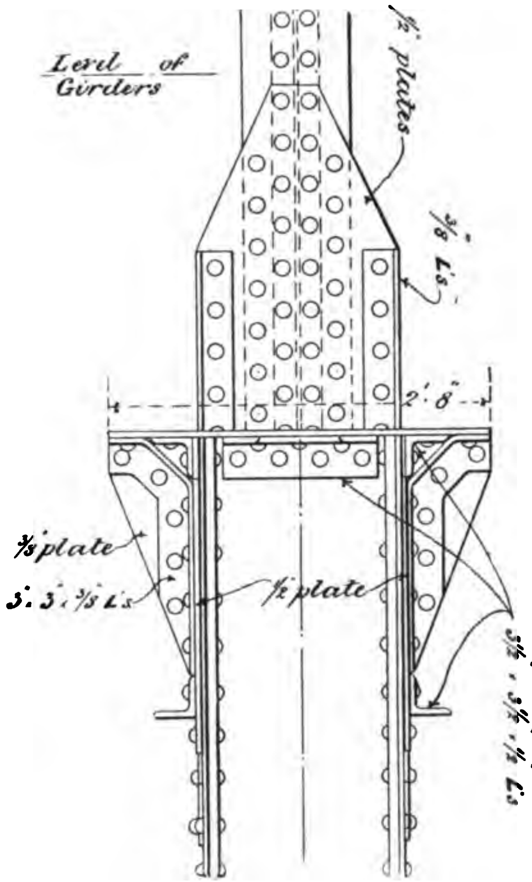


Fig. 209.
Scale ¾ inch = 1 foot.

the arrangement of the details and connections being determined by the conditions of the site and the peculiarities of loading.

It forms one of a row of columns dividing two adjacent bays of an extensive range of factories, boiler-shops, etc., the bay on one hand having a high-level traveller road, with high-level

roof, and on the other hand a low-level traveller road, with low-level roof. Together with the provision for these traveller and

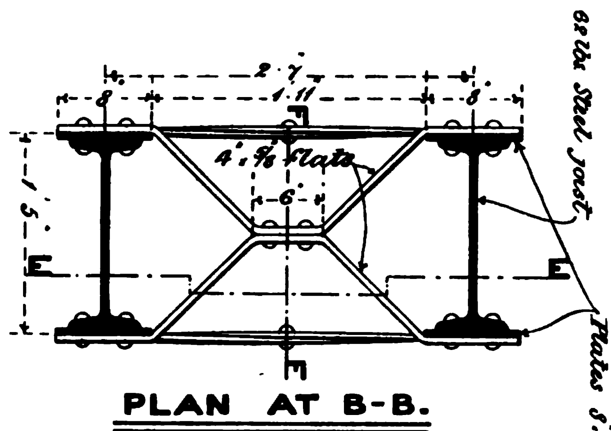


FIG. 210.
Scale ¾ inch = 1 foot.

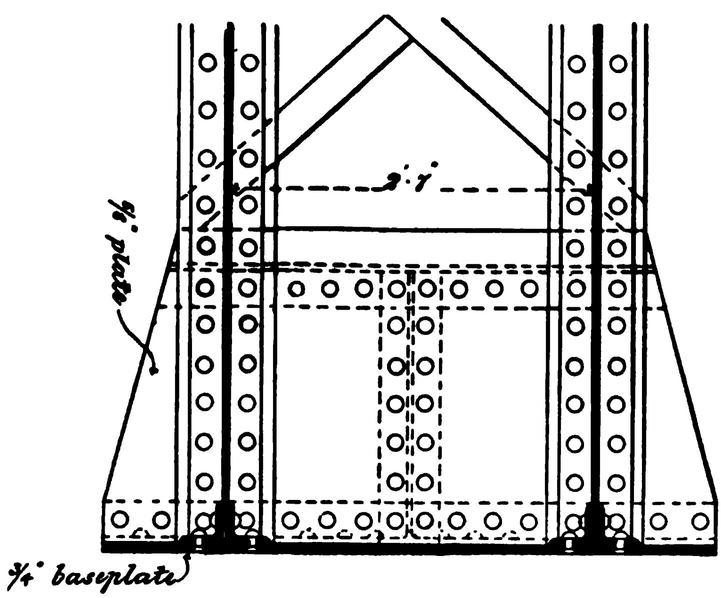


FIG. 211.
Scale ¾ inch = 1 foot.

roof loads a system of bracing is also attached for the purpose of carrying the various ranges of main and countershafting, with

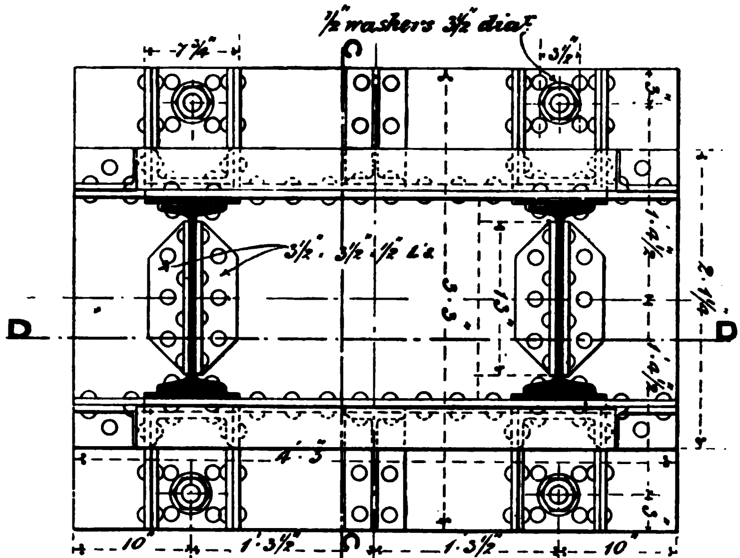


FIG. 212 (Scale 1/4 inch = 1 foot).

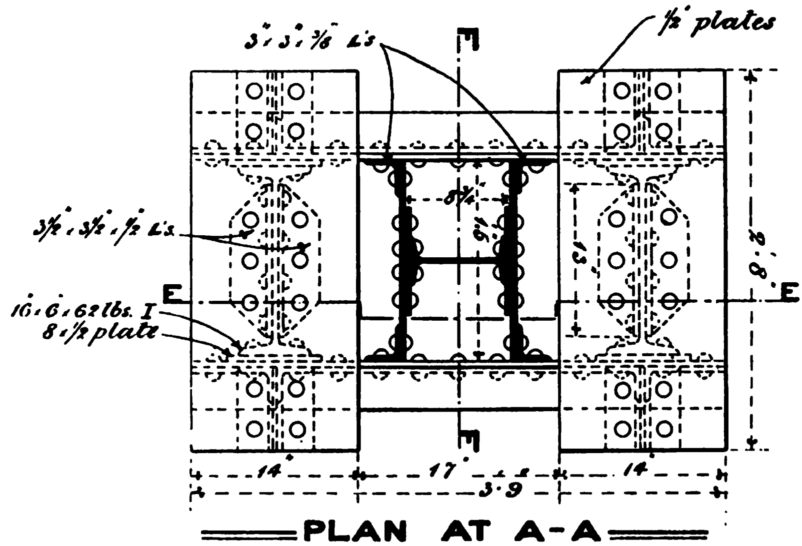


FIG. 213 (Scale 1/4 inch = 1 foot).

electric motors attached to the column, required for the varied machine tools to be used in the shops. The variety of loading and of detail of connection thus required rendered the design of these columns somewhat complex, and the results are not without their value to the student. The general arrangement of the columns,

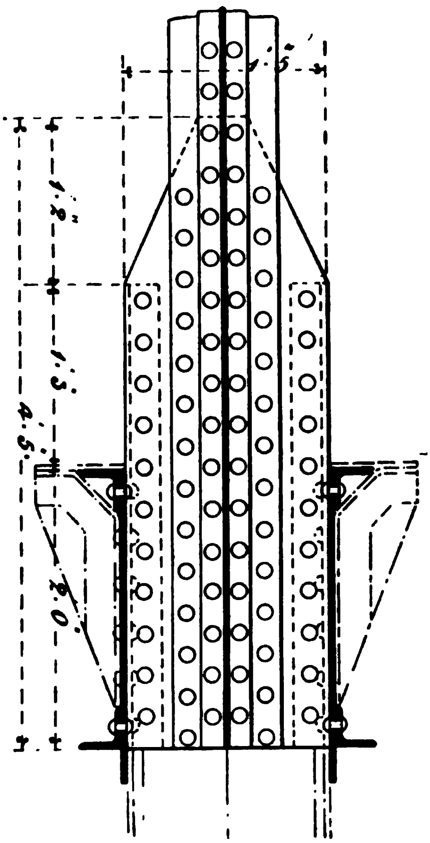


Fig. 214.
Scale ¾ inch = 1 foot.

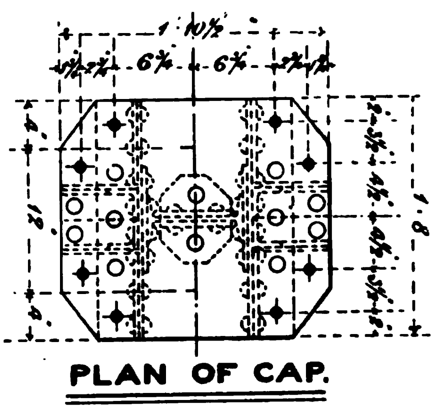


Fig. 215.
Scale ¾ inch = 1 foot.

and of the traveller girders, roof girders and roof principals associated with them, is shown in Figs. 216, 217, the bracing for countershafting, which supplied longitudinal stiffness to the rows of columns, being omitted for the sake of clearness.

Fig. 218 is a front elevation and Fig. 219 a side elevation of the base of the column. A plan of the base looking down is given in

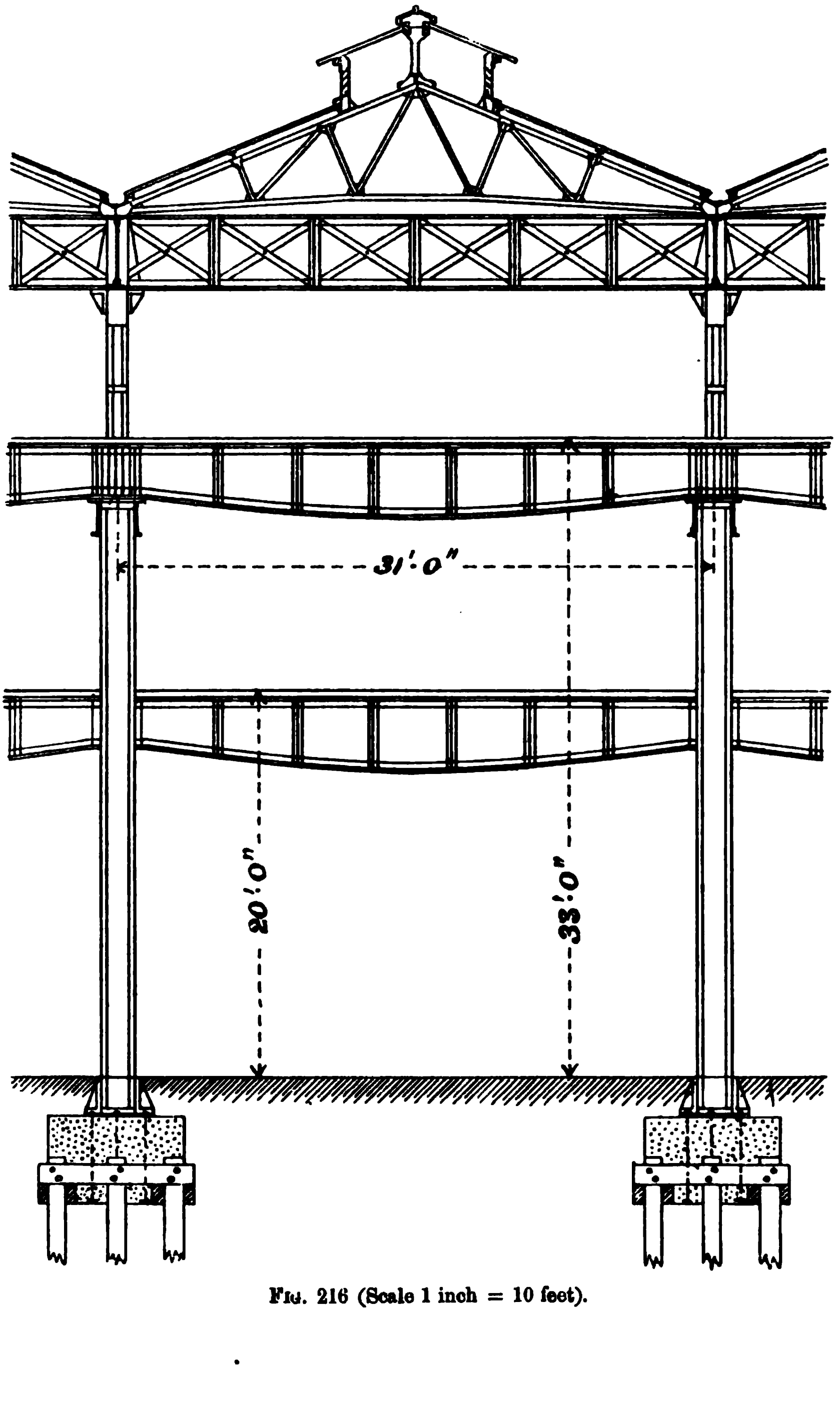


FIG. 216 (Scale 1 inch = 10 feet).

Fig. 220. It will be observed that the column is held down by eight foundation bolts, the details of which are shown in Fig. 221 and to which further reference will be made.

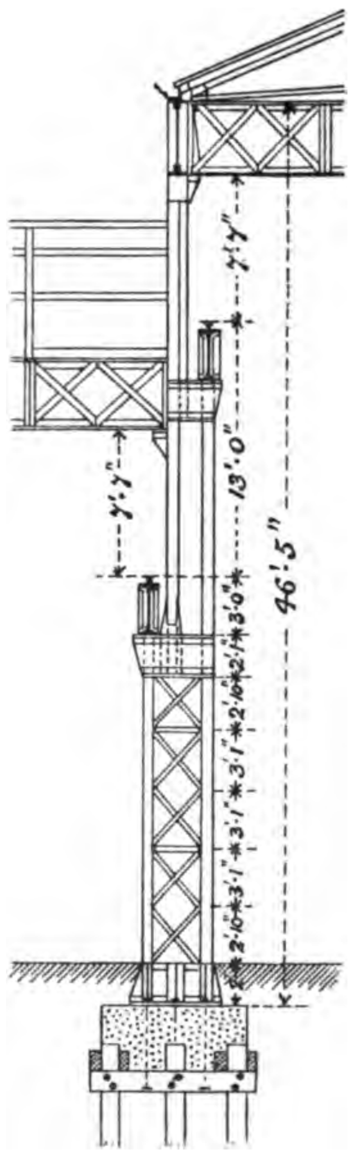


FIG. 217 (Scale 1 inch = 10 feet).

A longitudinal section of the base is also shown in Fig. 222.

The normal section of the lower portion of the column is given in Fig 223, from which it will be seen that the weight-bearing section consists of two girder-shaped riveted sections, each consist-

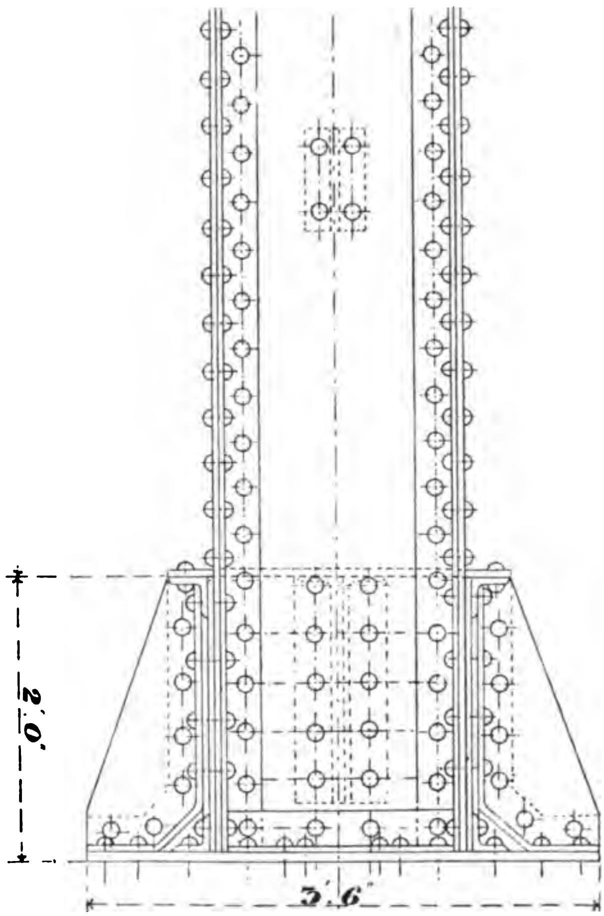


Fig. 218.
Scale $\frac{3}{4}$ inch = 1 foot.

ing of a solid web 20" × $\frac{1}{2}$", four angles $3\frac{1}{2}$" × $3\frac{1}{2}$" × $\frac{1}{2}$", and two plates 8" × $\frac{1}{2}$". The sections are prevented from twisting by a braced arrangement of flat bars 3" × $\frac{5}{8}$", arranged as shown, and spaced at intervals up the column, while the pair of girder sections

are united by a latticed web of angle irons riveted to the back of the girder webs as shown in Fig. 219, thereby converting the pairs

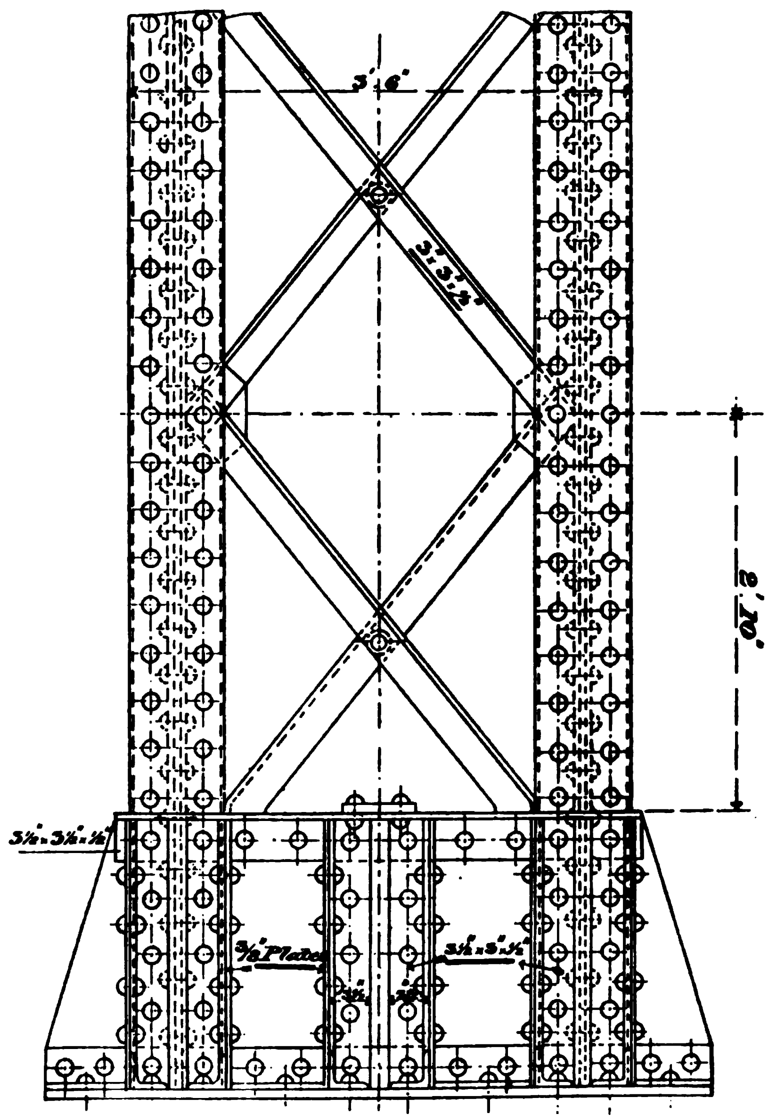


Fig. 219.
Scale ¾ inch = 1 foot.

of vertical columns into one braced vertical cantilever, capable of resisting transverse flexure, and with a very large moment of inertia in the plane of the latticed web; while in a plane at right angles thereto, and in the longitudinal axis of the traveller roads, the girder sections constituting the columns have each a large resistance to flexure in *that* plane.

This lower section of the column is continued up to the level of the lower traveller road, provision for which is made as shown in Fig. 224, one girder section being stopped off to carry the lower

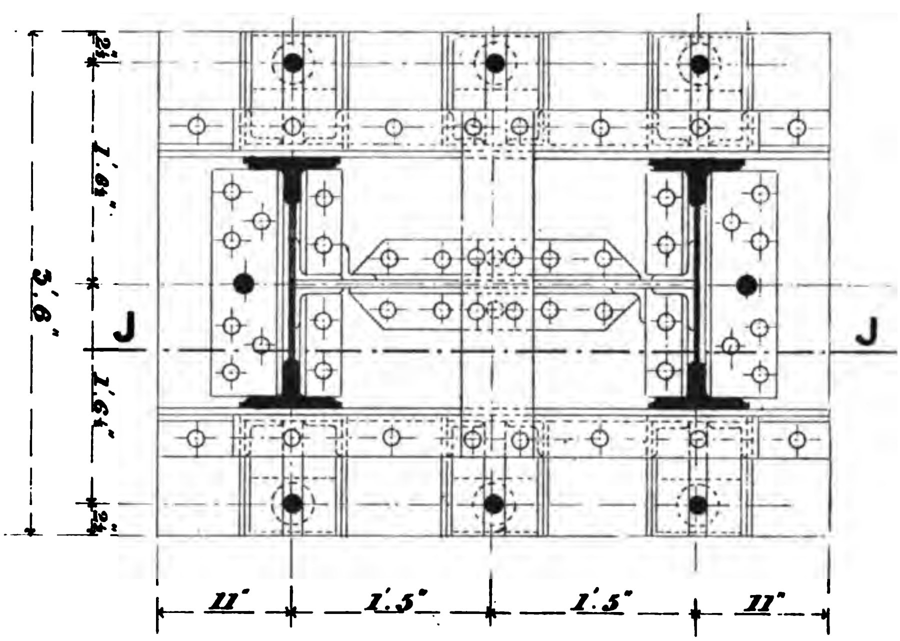


FIG. 220.
Scale ¾ inch = 1 foot.

traveller girder, while the other girder section is continued up unbroken to carry the higher traveller as shown in Figs. 217, 225.

It will be seen in Fig. 224 that at the level of the lower traveller seating a new vertical member commences, intended to carry the roof loading above. The base of this roof column is arranged to rest upon the arrangement of plates and angles shown in Fig. 224, by which means the roof load is divided between the twin girder sections of the lower portion of the column. Referring again to Fig. 225, we see that while the column section

is stopped at the level of the upper traveller road, the roof column is continued upwards to the summit of the entire column, the total height from the under side of the column being 46 feet 5 inches. The section of the column between the levels of the lower and upper traveller ways is shown in Fig. 230.

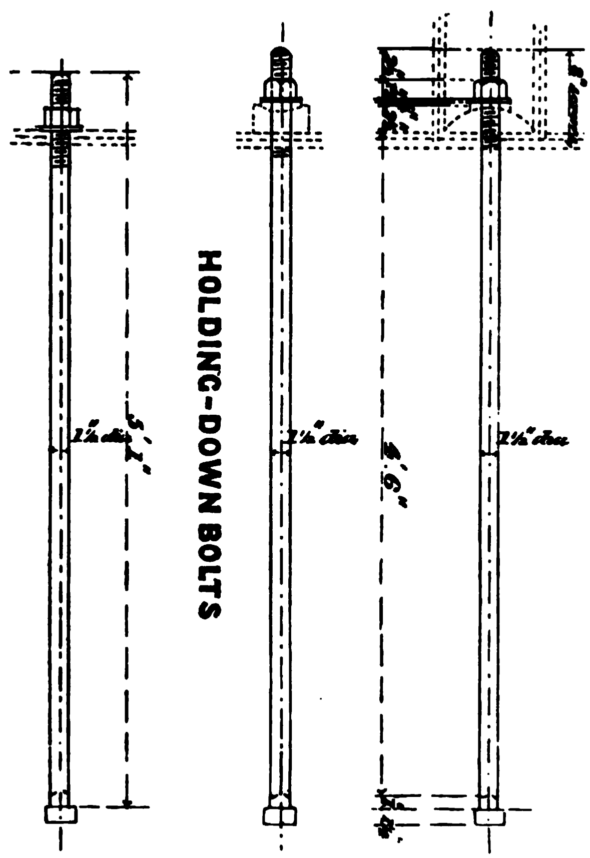


FIG. 221.
Scale ¾ inch = 1 foot.

The student will observe that the clearance between the faces of columns and the centre line of traveller rail is in this case 11 inches, and he will be in a position to appreciate the influence which this dimension has on the arrangement of details in cases of this kind.

Fig. 226 is a sectional plan on line FF at the level of the lower traveller girder seating shown in elevation in Fig. 224, while

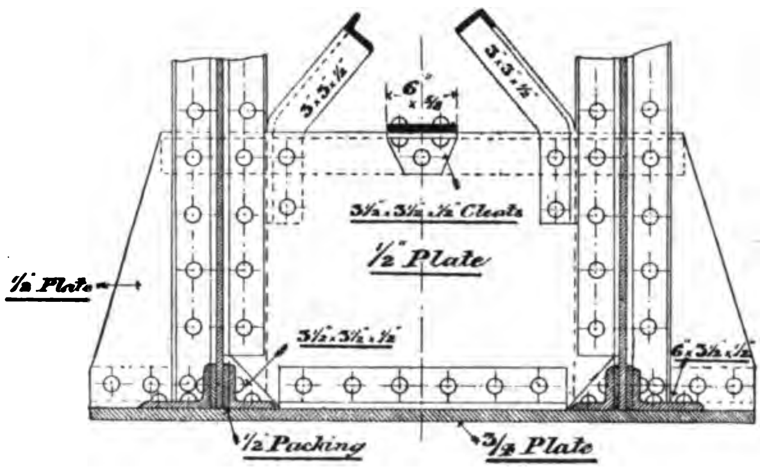


FIG. 222.
Scale ¾ inch = 1 foot.

Fig. 227 is a section on GG, Fig. 224. Fig. 228 is a section through DD (Fig. 225), and Fig. 229 is a section at CC, looking down upon the seating of the upper traveller girder.

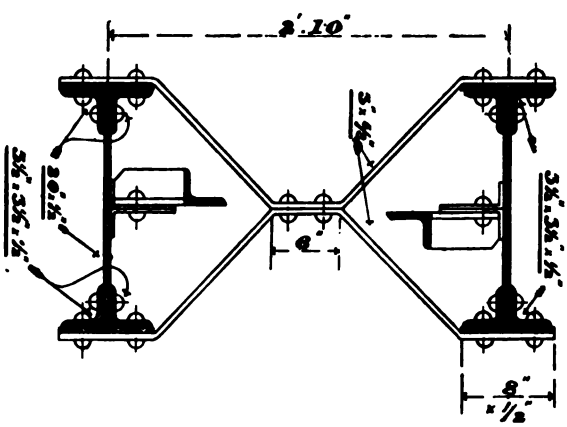


FIG. 223.
Scale ¾ inch = 1 foot.

The uppermost portion of the column above the upper traveller road, carrying the lattice roof girder, has sections, which are shown

in Figs. 231 and 232, while the detail of the bolted connection between column and roof girders is shown in Figs. 85, 86. The details of the girders themselves are alluded to, together with those

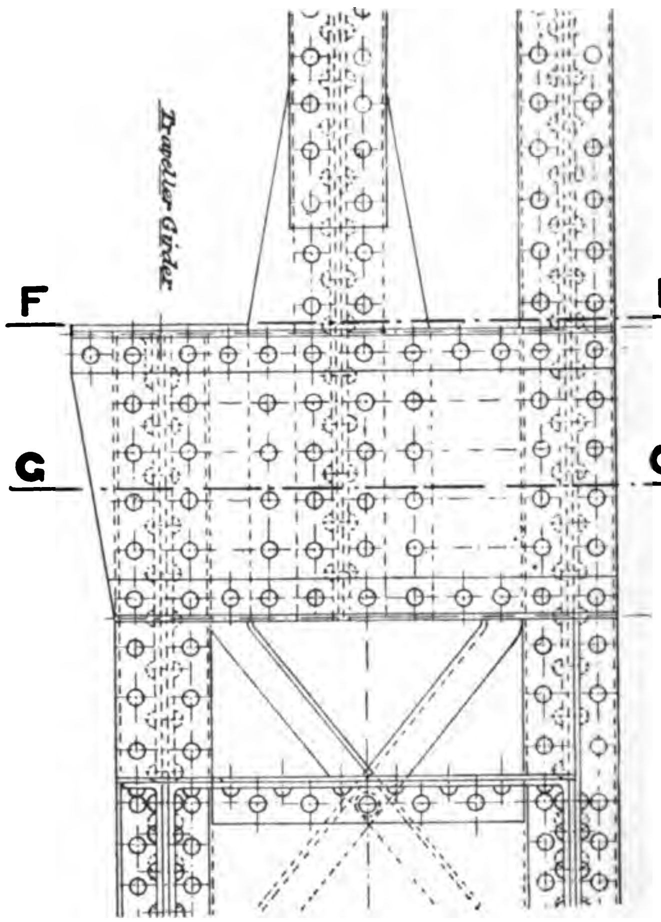


FIG. 224.
Scale ¾ inch = 1 foot.

of the roof principals which they support, in Chaps. III. and V., pp. 149 and 313.

The general question of foundations is not one which comes within the immediate scope of these notes, but a few remarks upon the nature of the foundations required for the column above described may not be inappropriate.

The column in question was built upon made ground, and experimental tests of its weight-bearing capacity showed that the load to be imposed could not be carried by the bare soil without

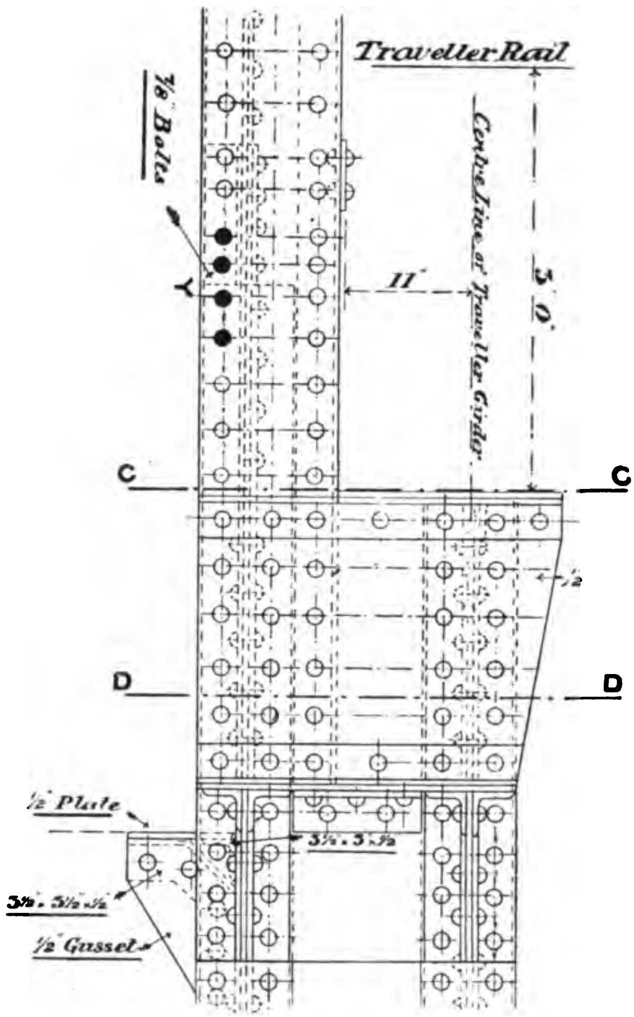


FIG. 225.
Scale ⅜ inch = 1 foot.

risk of an amount of settlement which might prove injurious to the traveller roads and the main and countershafting. Piled foundations were therefore provided for the column bases in the

manner indicated in Figs. 233, 234 and 235, in front and side elevation and plan.

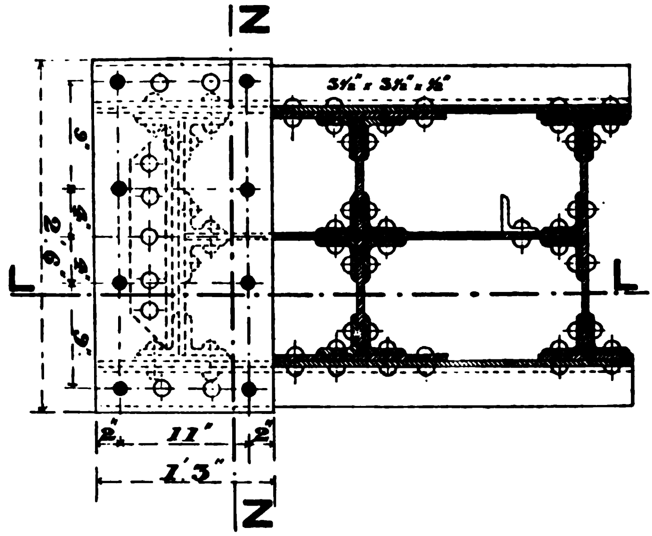


Fig. 226.
Scale ¾ inch = 1 foot.

A group of eight piles of whole timber driven down to the solid is connected at the top by a grillage of half-timber sills

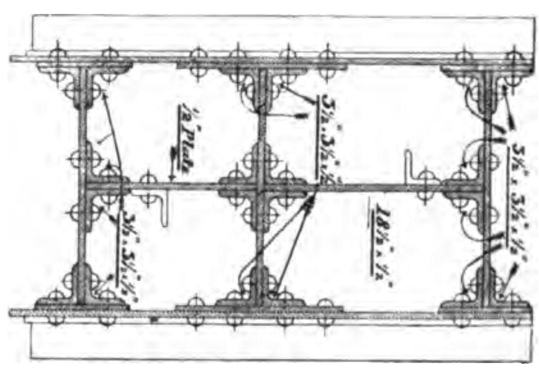


Fig. 227.
Scale ¾ inch = 1 foot.

bolted to the piles as shown, the whole being capped by a massive bed of Portland cement concrete, with carefully levelled top surface,

upon which the bases of the columns shown in Fig. 216 were planted.

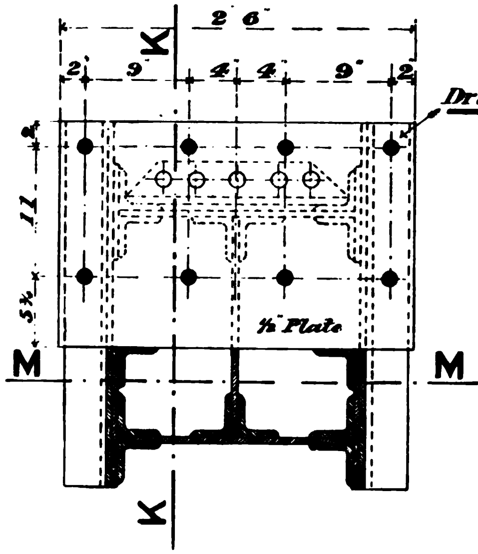


FIG. 229.
Scale ¾ inch = 1 foot.

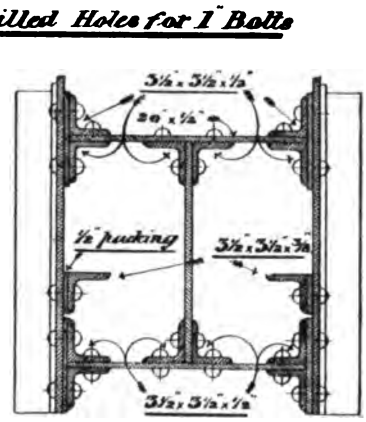


FIG. 228.
Scale ¾ inch = 1 foot.

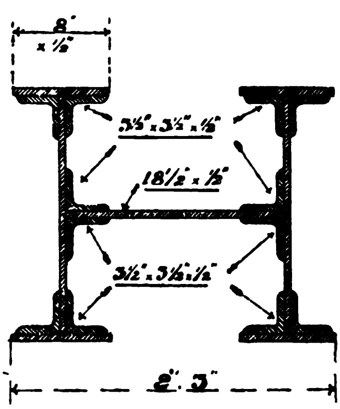


FIG. 230.
Scale ¾ inch = 1 foot.

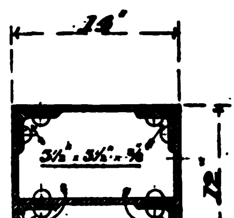


FIG. 231.
Scale ¾ inch = 1 foot.

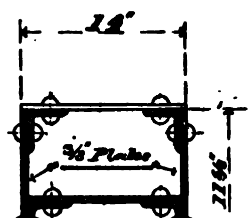


FIG. 232.
Scale ¾ inch = 1 foot.

The general arrangement of the building is such that any overturning moment due to horizontal wind pressure acting at the

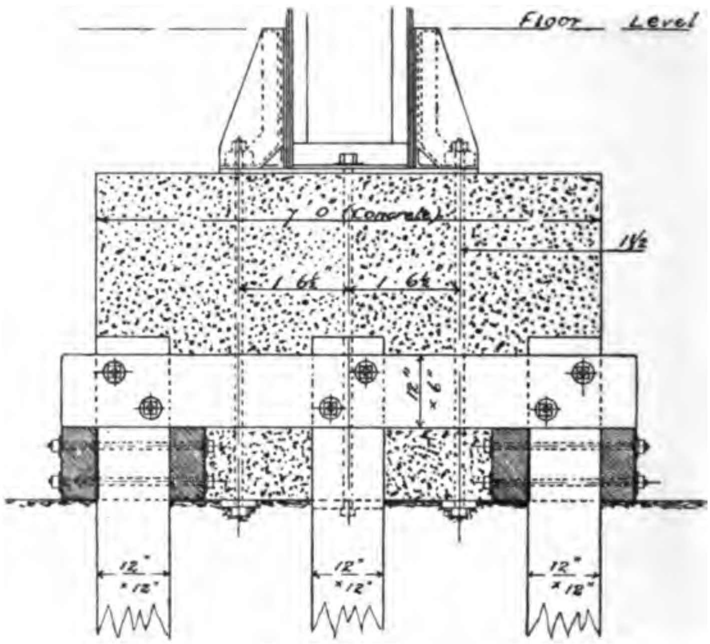


Fig. 233 (Scale [illegible] inch = 1 foot).

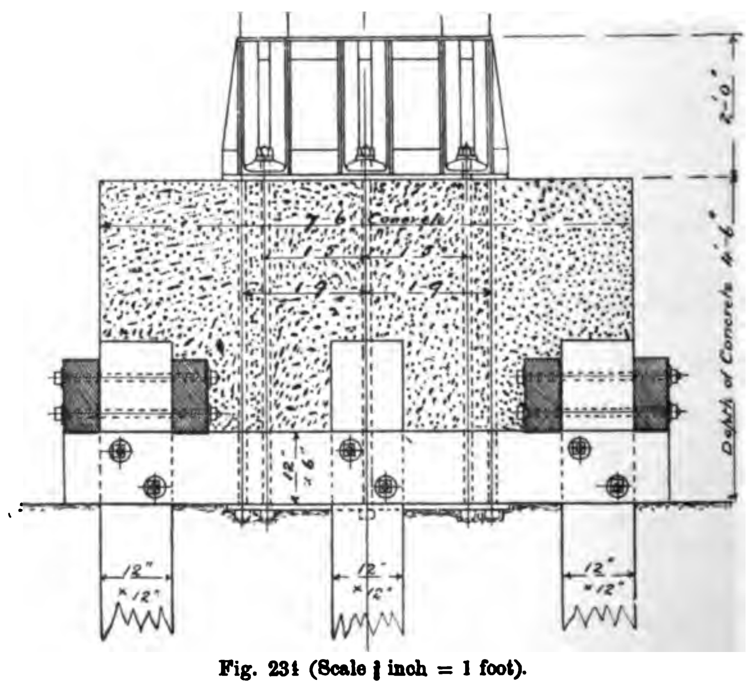


Fig. 234 (Scale [illegible] inch = 1 foot).

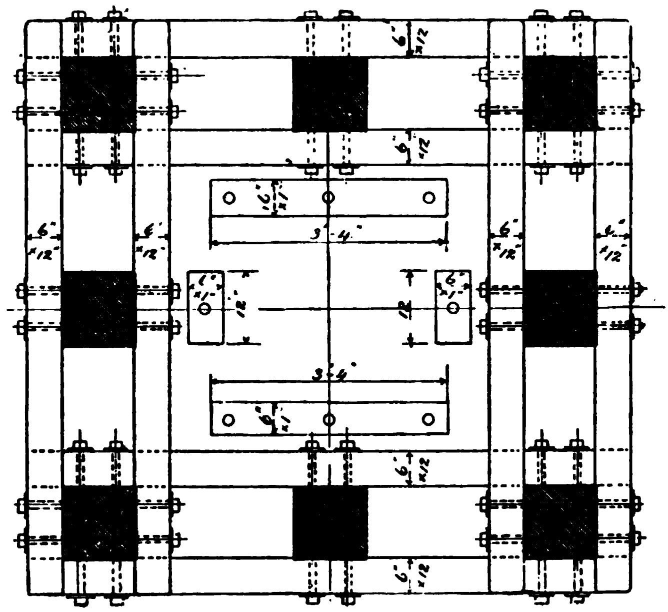


Fig. 235 (Scale $\frac{3}{8}$ inch = 1 foot).

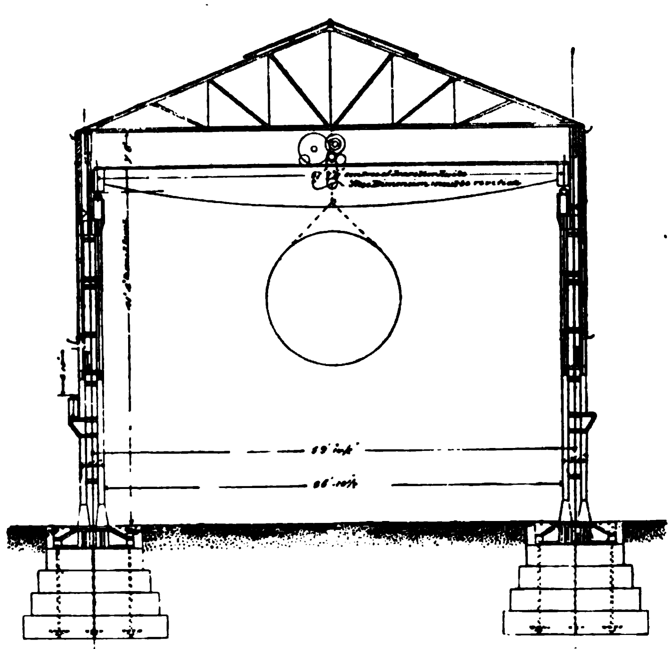


Fig. 236 (Scale 1 inch = 24 feet).

top of the column is divided between several rows of columns, the whole being enclosed between masonry walls of a substantial character. Consequently the amount of holding-down power

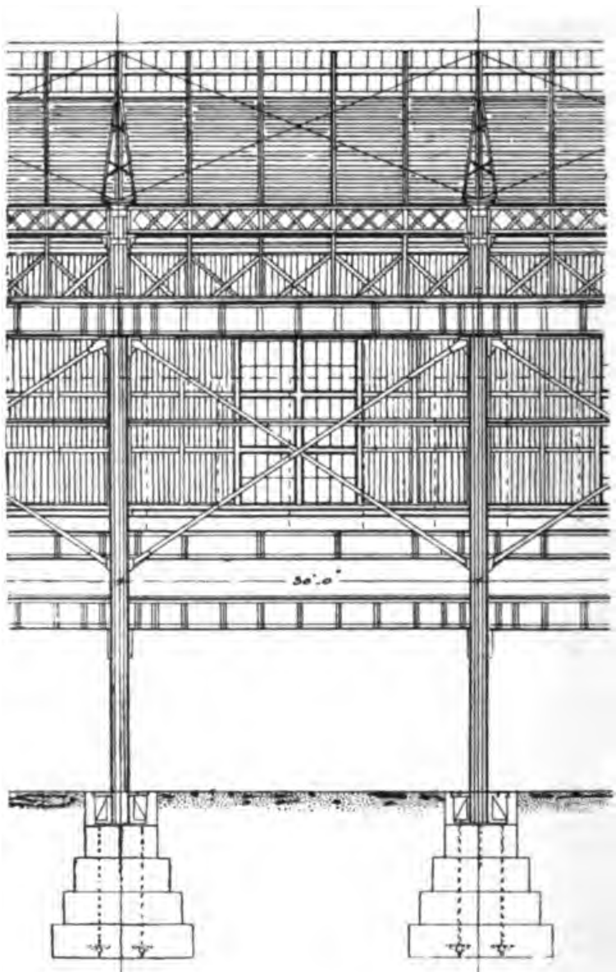


FIG. 237.
Scale 1 inch = 16 feet.

required to resist such a moment was not excessive, and the group of eight foundation bolts shown in detail in Fig. 221 provided a sufficient resistance. The arrangement of these bolts

is shown in plan in Fig. 235, and it will be observed that their washers, in this case, consist of simple steel flats, 1 inch thick, arranged as shown.

Under other conditions of construction, however, the question

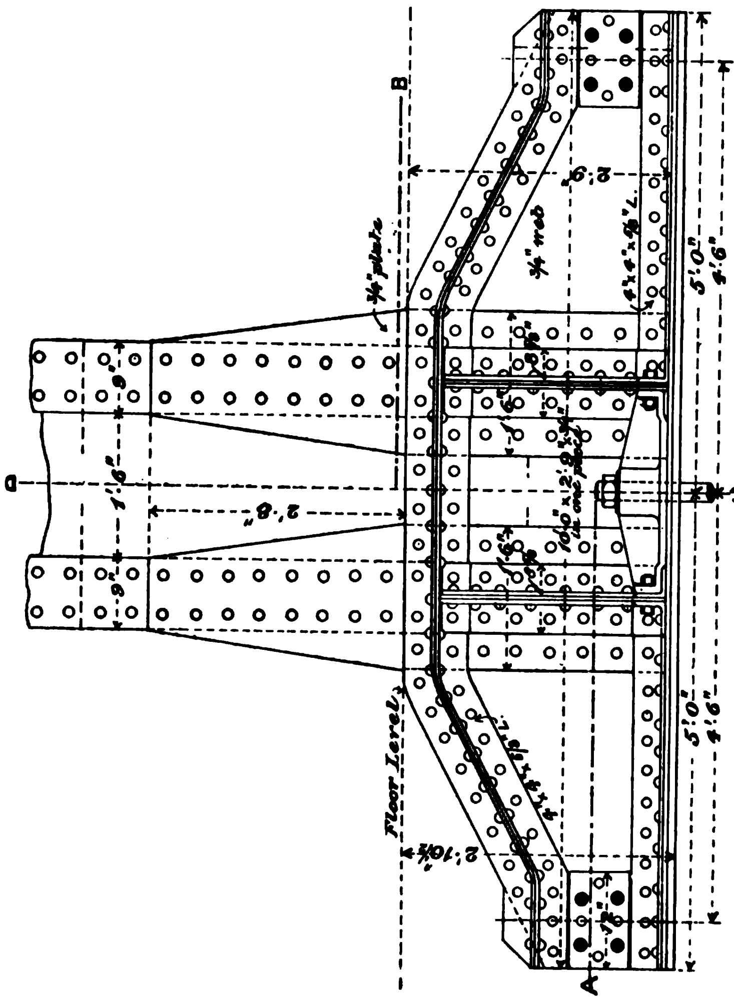


FIG. 238.
Scale 1/4 inch = 1 foot.

of lateral stability of the whole structure against wind pressure may be of greater importance, and require special provision to ensure sufficient resistance to an overturning moment.

Especially may this be the case where the building is lofty

and the enclosing walls composed of timber or corrugated sheet-iron, or both combined, having no great weight in themselves, and therefore requiring a sufficiency of stability in the column anchorages.

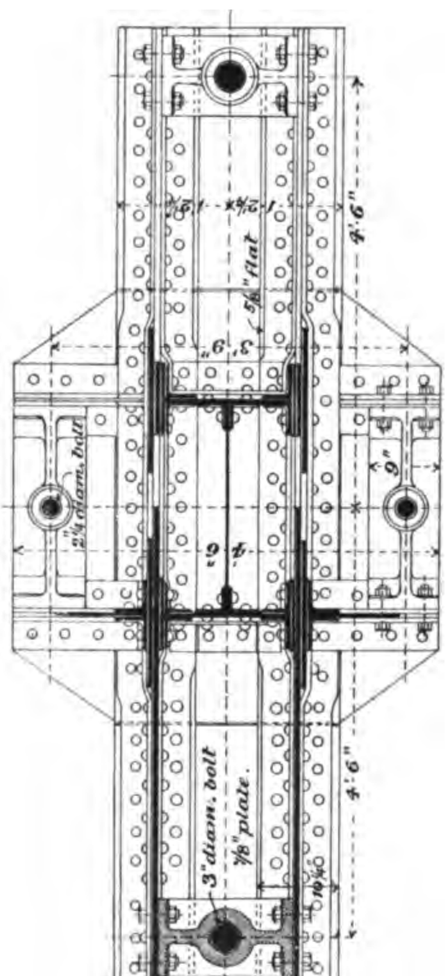


FIG. 239.
Scale ½ inch = 1 foot.

An instance of this class is illustrated in Fig. 236, showing a cross-section of a building designed to carry 70-ton travellers at a height of rail level of 41 feet 4 inches above ground, the elevation of one bay of the building being shown in Fig. 237.

In this case the columns of riveted wrought iron rest upon massive concrete foundations, carried down to a reliable stratum underlying soft alluvial deposit.

The section of this column, which is 49 feet in total height, is of the type shown in Fig. 167, and the details of the base and anchorages are shown in Figs. 238, 239, 240, 241, 242, and 243. It will be seen that the holding-down bolts resisting overturning

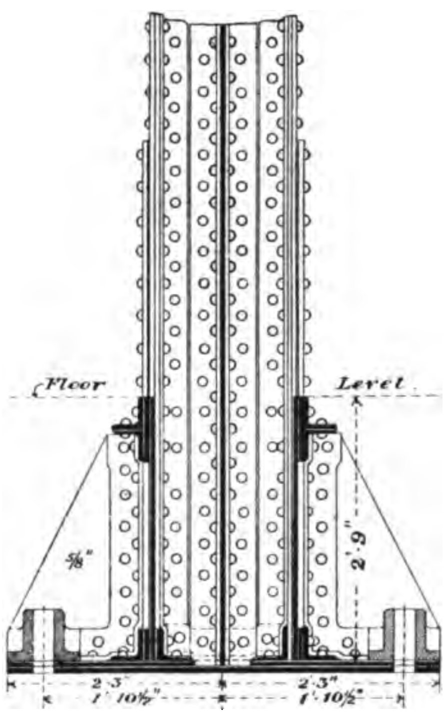


FIG. 240.
Scale ¼ inch = 1 foot.

moments transversely to the building are 3 inches in diameter, of mild steel, with circular cast-iron washers, while sufficient stability in the longitudinal direction of the building is afforded by bolts 2¼ inches in diameter.

The attention of the student is directed to the means by which the heavy anchorage bolts take hold upon the riveted column base.

The detail of the cap of the column and the seating provided for the traveller girders is given in Fig. 244, and the section of the traveller girders showing elm timber sleeper forming continuous

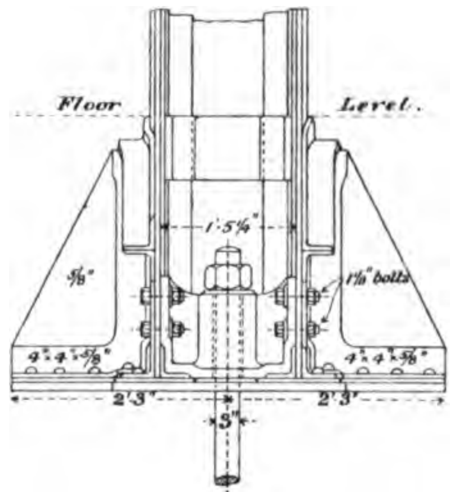


FIG. 241.
Scale ¼ inch = 1 foot.

bearer to rail in Fig. 245, the timber being notched to the stepping up of the plates in top flange, the traveller girder in this case being of uniform depth, and not fish-bellied.

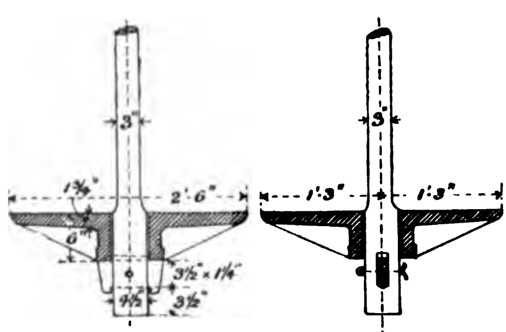


FIG. 242.
Scale ¼ inch = 1 foot.

In such a case as the foregoing, the fixing of heavy anchorage bolts during erection requires careful attention to ensure that the bolts themselves do not sink beyond their true level when concrete

is being deposited around them, as in such an event the screwed portion of the bolts may be found insufficient in length to pass

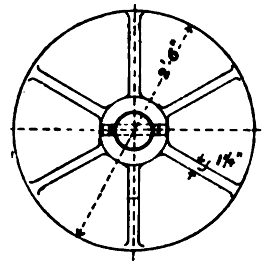


Fig. 243.
Scale ¼ inch = 1 foot.

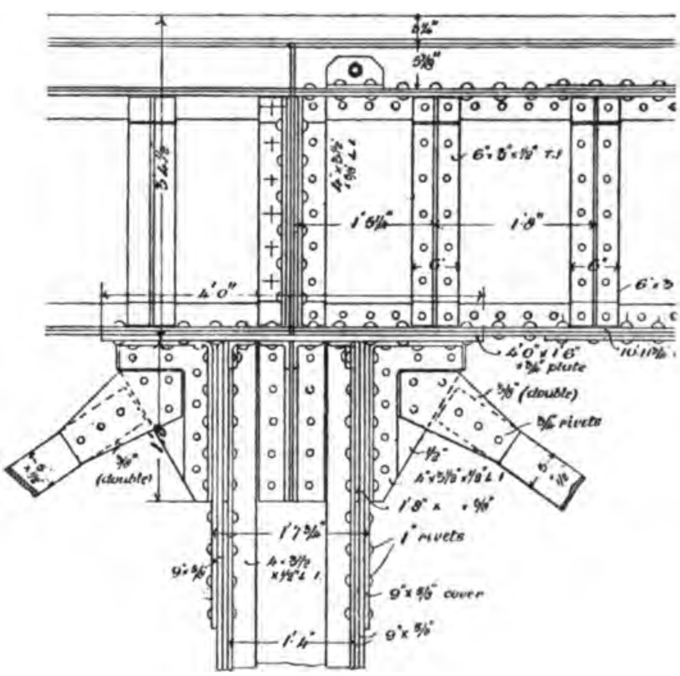


Fig. 244.
Scale ¼ inch = 1 foot.

through the whole depth of the nut, a contingency which points to the desirability of an ample margin of length in the bolt and its screwed portion, the labour of cutting off excess length in such

conditions being preferable to any loss of strength in the hold on the nut.

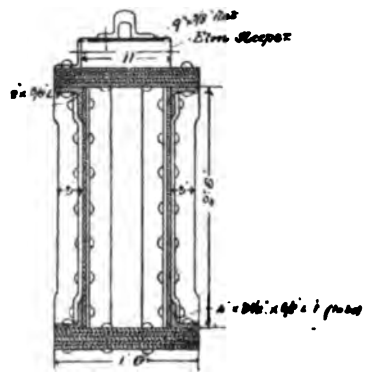


FIG. 245.
Scale ¼ inch = 1 foot.

The accuracy of position of all anchorage bolts in plan should be secured by the use of templets.

CHAPTER V.

ROOF CONSTRUCTION IN MILD STEEL AND IRON.

General remarks—Development of roof construction in timber, cast iron, wrought iron, wrought iron and steel, mild steel—Classification of roof principals—Members of roof principals—Upper or compressive member or principal rafter—Sections for principal rafter—Shoes to rafter—Expansion apparatus—Main tie or lower tension member; in timber roofs; in composite roofs; in wrought-iron roofs; in steel roofs—Risks of defective smith-work—Earlier steel tie-rods—Present-day practice—Flat bar ties—Link tie-rods—Occasional stiffening of main tie-rod in small roofs—Examples of tie-rods—Intermediate bracing—Struts—Ties—Purlins—Influence of nature of roof covering upon the arrangement of purlins—Details and sections of purlins—Distance apart of main trusses—Intermediate rafters—Roofing accessories—The collection and disposal of rainwater or melted snow—General arrangement of roof drainage—Roof guttering in cast iron or riveted steel; in lead—Experiment on rate of discharge in gutters and cesspools—Area of roof surface to be drained—Examples of guttering and down-pipes—Expansion joints—Stopped ends—Lanterns, skylights, and ventilators—General remarks—Lantern standards—Louvre blades—Roofs of flat pitch—Examples of roof construction of various types—Special type of roofing combined with vertical supports—Details—The testing of roof principals—Conditions of practical testing in the contractor's yard—Methods of measuring deformation and settlement—Remarks on cottering up—Setting out of roof principals—Scribing floor.

It is impossible, within the limits of a single chapter in a collection of "Notes" such as the present, adequately to deal even with the main features of roof construction in mild steel or iron.

All that the writer can hope to accomplish is to offer such suggestive remarks on the subject as may assist the student to a fuller consideration of this branch of practical construction, and lead him to a careful study of the numerous existing examples.

It will be assumed that the student has made himself acquainted with the usually accepted theories as to wind pressure, and the conditions of loading of a roof truss arising from dead load of structure, weight of snow, and wind pressure, whether the latter be considered as acting vertically, normal to the slope of

the roof, horizontally, equally or unequally distributed; and that he is acquainted with the usual methods of calculations of stresses, such as graphic analysis and the method of sections.

Nor can the extensive subject of the nature and properties of the various kinds of roof coverings, such as slate, tiling, glass, zinc, copper, lead, corrugated iron, felt, and the like, be considered, except so far as they may influence the arrangement and detail of the metallic structure which is designed to support them.[1]

The history of the development of roof construction from its earlier forms in timber, and through the further stages of cast iron, wrought iron, with various cast-iron details, then wrought iron practically alone, wrought iron with steel tie-bars, and lastly, as at the present time, in mild steel, with occasionally some admixture in detail of the other two metals, would doubtless be both interesting and instructive, but practical consideration can only here be given to the final stage of mild steel.

Roof principals may be roughly divided into four main divisions, viz.:—

(*a*) Principals with straight upper rafters, of varying degrees of pitch.

(*b*) Principals with curved or polygonal upper rafters.

(*c*) Principals of special constructions, including the arch, arch with one, two, or three hinges, or of the "sickle" type indicated in Fig. 345, usually employed only in large spans.

(*d*) To the above may be added another class sometimes employed in covering large areas, in which lattice girders with parallel booms, and sometimes of large span, are placed side by side, and roofed over with intermediate principals, usually of the first type described above, but occasionally of a special class. In this class the upper boom of the main lattice girder supports a valley gutter for its entire length, or a series of ridges and valleys is arranged to cover the intermediate space.

A further subdivision may also be taken to include a type of roof covering where the conditions require a cantilever method of construction, such as large verandahs, or the form frequently

[1] The student will find valuable assistance in the subjects of roofing materials in Vol. I. and in the Calculation of Stresses and the Theory of Roof Loading in Vol. IV. of "Notes on Building Construction."

met with in railway station platforms, where the position of the supporting columns at some distance from the edge of the platform necessitates the continuation of the roof truss in the form of a cantilever. In all such cases the deflection of the cantilever portion must be carefully borne in mind, in order that the construction, guttering, etc., at the eaves may maintain their true and horizontal lines.

Another arrangement of roof principals has been suggested, dividing them into two classes, intended to cover all roof structures, namely, the one in which the reactions of the supports of the principal are in a vertical direction, and the other in which the reactions are at an angle with the vertical, the first class being self-contained without horizontal thrust, the second those with a horizontal thrust, and dependent upon the resistance of the abutments for their stability.

This distinction would, however, appear to fail unless the loading of the principal is purely vertical in both classes. Where the assumptions as to wind pressure include a horizontal component, as in the case of wind pressure taken normal to the slope of the roof, other considerations present themselves, and the reactions of one or both supports, even in self-contained structures, must contribute a corresponding and opposing force.

Roof principals of the class (*a*) or (*b*) present three main features in their design, viz. :—

The upper or compressive member, usually denominated the principal rafter, either straight, curved, or polygonal, as the case may be.

The lower or tension member, denominated the main tie, or, in timber roofs, the tie beam.

The intermediate bracing of struts and ties, fulfilling similar functions to those of the web of lattice girders.

The upper or compressive member, or principal rafter, will have its scantlings determined in the first instance by the laws governing the strength of long columns or struts, the length of the column under consideration being determined in a vertical plane by the distance between the apices or points of junction of the intermediate bracings. In a longitudinal direction, however, the column will be free to deflect laterally between the points of support of the purlins, assuming the latter to offer a sufficient resistance to lateral flexure of the principal as a whole; but, as remarked further on, roof principals in course of erection or

testing are, in the temporary absence of purlins or roof coverings, somewhat flexible in a plane at right angles to their elevations, owing to the smallness of their dimensions in that plane as compared with their span. Security in this respect is obtained by a properly designed system of what is called wind bracing, being an arrangement of diagonal braces from the heel or shoe of one principal to the ridge or summit of another, whereby the possibility of the overturning of a series of roof principals like a pack of cards is obviated. Where the length of roof is not great, and the roof is enclosed between stout gable walls at the ends, or is hipped, the addition of wind bracing is not so imperative.

It will be found, however, in practical design, that the scantlings of the principal rafter will be ruled by other considerations than those of columns or strut area alone, even if the compressive stresses be purely axial, in the direction of the length of the column. If, on the other hand, the column is subjected to transverse stress arising from the position of the purlin not being precisely over the junction of a brace, a condition which will frequently arise in roofs of small span, then the stresses arising from the bending moments must be considered in connection with those arising from purely compressive stress, and the area or moment of inertia of the section increased accordingly.

The construction of skylights or ventilating lanterns with standards attached to the principal rafters, examples of which will be given later on, will frequently influence the choice of section, and impose a minimum dimension in order that the bolted or riveted attachments may be properly made. Thus, for example, a tee-steel section for the principal rafter may be selected, giving a sufficiency of area for the calculated stresses, but the top table of which may be too narrow to receive the bolts required to connect a cast-iron louvre standard of the type shown in Fig. 265.

Or again, the section may not be suitable to properly arrange the details required at the connection of the rafters at the apex of the principal.

Mistakes in points of detail such as these (upon which much of the success in design depends) will be usually avoided if the student is careful to draw each detail in cross-section as well as in elevation.

The details of connection of the purlins with the principal

rafter must also be considered and allowed for. Where wind bracing is adopted, the scantlings required for connection to the top table or web of the rafter must be remembered.

The sections commonly used in the construction of the principal rafter are various, and adapted to the span, distance apart of principals, load, and working stress allowed.

Thus for roofs of small span, and where precise symmetry about a central axis is not necessary, a single angle (Fig. 142) may be used. For roofs up to about 40 to 50 feet span a single tee section (Fig. 147) is commonly adopted. A section of double angles (Fig. 143) is very convenient for connection, and affords more space for bolted or riveted details in the top tables. A built-up tee section of plates and angles is convenient for larger spans, as in Fig. 146, with the addition of a vertical web plate.

A section of double channels (Fig. 150) has been used for spans of from 90 to 100 feet, while the built-up channel sections shown in Fig. 163, with flat bar lattice bracing, have been used in a roof of about 120 feet span. Roofs of still larger spans, up to 200 feet or more, have been constructed with principal rafter sections of the types shown in Fig. 151, or in Fig. 162, with additional flange plates top and bottom.

The details of the lower or shoe end of the upper rafter are variable in character, depending largely on the span of the principal, and the scantlings of the main tie. If this latter is of heavy section, the connection at the heel of the principal becomes of corresponding importance, and demands careful consideration.

For roofs of moderate span, say up to 60 feet or thereabouts, the lower end of the principal rafter terminates in, and is connected to, a shoe which forms the seating of the principal upon the wall, column, or girder, as the case may be. Formerly this shoe was of cast iron of various forms, and some defective details may be discovered in those forms of shoe in which the method of connection of the main tie involved tension upon certain portions of the cast iron. In present-day practice these cast-iron shoes have generally been superseded by shoes of a simple form in mild-steel riveted work. A few examples of shoes of this type, with their connection to the main tie, are shown in Figs. 280, 286 to 293, 298 to 302.

In roofs of large span the expansion and contraction of the structure under changes of temperature have to be provided for and in these cases the place of the ordinary shoe is frequently taken by a system of rocker plates and rollers resembling the

ordinary expansion apparatus of a girder of large span, although it may be questioned whether such rollers, not easily accessible as a rule to inspection, do not frequently become rusted up to an extent which interferes with their efficiency.

The Main Tie or Lower Tension Member.—The form of section to be given to this important member of a roof-truss, especially of the classes (*a*) and (*b*) above alluded to, will largely influence the details of connections and the general type of construction, and will always be found to demand careful consideration.

In timber roofs this member takes the form of a simple rectangular beam, as the functions of this tie are usually as much to resist transverse stress due to the weight of ceiling rafters, or possibly of a floor, as to resist in tension the spreading effort of the rafters.

In composite roofs of timber and iron combined the practice has usually been to employ wrought-iron tie-rods of circular section.

In wrought-iron roofing for moderate spans, and even up to spans of very considerable dimensions, the general practice for many years was in favour of the round rod or circular section.

This form admitted of a nice adjustment of cross-sectional area to theoretical requirements, and so far represented an economical construction, while the appearance was light, and the nature of the material offered no special difficulties or risks in the manufacture of eyes and jaws or in the welding processes which usually accompany those details, provided only that the smith-work was properly and soundly done.

In many roofs of large span heavy circular rods were used with screwed ends and coupled connections, with a view probably of avoiding the risks of defective smith-work in such large jaws or eyes as the size of section demanded.

Recent events have, however, thrown some light upon the general policy of providing a single member only to act as the main tie in roofs of large span, and it may be doubted whether, in view of possible hidden flaws arising from defective smith-work, a duplication of this important member in a large roof-truss is not desirable.

Upon the introduction of steel into roof construction some attempts were made to utilize the material possessing a high tensile resistance, by employing it in the main tie, while constructing the remainder of the truss in wrought iron. In the early days

of steel construction a harder grade of steel, with higher carbon content, was in vogue, and difficulties manifested themselves in the smithing of jaws and eyes. In some cases the difficulty was met by the use of a ferrule of soft wrought iron being welded on the steel rod, and the whole smithed out to the desired form; while in other directions steel tie-rods with plain screwed ends, involving no smith-work or welding, were used, the connections being of a special character, consisting of coupling boxes designed to give the required connections at junctions with intermediate braces, etc.

With the more general use of mild steel of a lower carbon content, and more amenable to smith-work, a new set of conditions has arisen, and the present-day practice is in favour of the use of mild steel throughout the entire truss, with such occasional use of cast iron in special details, such as lanterns, skylights, and guttering, as is desirable. Some designers, however, prefer, in the case of round-rod ties, to use wrought iron, on the ground of greater security in the welds, reserving mild steel for the remainder of the truss, thus reversing the procedure of an earlier date; but the tendency as a rule has been to abandon the circular section, and to adopt for moderate spans the flat-bar tie with riveted connections.

This form of tie is less economical than the circular section, inasmuch as a loss of one rivet-hole in the cross-section is involved; the appearance, although not objectionable, is not so light as the round rod, but, on the other hand, the method of construction is cheap, involving no smith-work, while the riveted connections are as a rule of simple type. If the slight increase of weight in the flat-bar type is set against the greater economy in manufacture, it is probable that the difference of cost in the two types, flat-bar section and circular section, is not very appreciable.

In roofs of large span, the ordinary suspension-bridge link with swelled eyes and pin connection has been employed with good results both in wrought iron and steel.

In this case all the precautions necessary, both as to methods of manufacture and in the design of the proper shape of head and dimensions of pin connection, are as applicable as in the case of suspension-bridge design, or in the lower chords of trussed girders, with eye-bar tension members.

Hitherto we have regarded the main tie as subject to direct tension only, and this assumption is probably correct for all roofs of large span and of considerable dead load in proportion to any

inequality or obliquity of loading or wind pressure which may come upon them. Certain cases may, however, arise, especially in roofs of small span in very exposed situations, where it is expedient to stiffen the main tie as against any small element of compression which may arise from an extreme horizontal component of wind pressure, or where the roof principal performs the function of a strut or tie between the heads of lofty columns, acting as a gauge-keeper between the parallel rails of a traveller gantry, or transmitting a proportion of wind pressure in a lofty building from one side to the other. Again, such a stiffening of the main tie may be desirable in the case of roof principals spanning the interval between lattice roof girders possessing but little transverse stiffness, and where a certain amount of stiffness in the tie-rod is desirable on general grounds.

Considerations of this kind will occasionally lead to the adoption of angle or tee, or other stiffened section, for the main tie, although, of course, the economy of section as for a purely tension member is lost, owing to the practical difficulties in connection with end connections, which lead to an inequality of tensile stress over the entire cross-section.

It is unnecessary to remark that where a tie is subjected to transverse stress, as from supported loads, the weight of a ceiling or floor, or the like, then the form of section must be one specially adapted to meet these conditions.

Some examples of the use of the circular section of tie-rod, with the details appertaining thereto, are given in Figs. 290 to 295.

The treatment of flat-bar ties in mild steel is indicated in Figs. 280 to 289, and of the stiffened form of tie in certain cases in Figs. 270 to 279.

The Intermediate Bracing of Struts and Ties.—The design of the struts forming portion of the intermediate bracing in trussed principals will be governed by the laws of long columns, and they will generally be found to be free from the transverse stresses which may sometimes affect the sectional area required in the main rafters; but, considered as columns, due allowance must be made for the imperfect seating or fixing of the ends of the struts, due to the exigencies of design in certain types of construction. Where a *boná-fide* pin end can be obtained at both ends of the strut, the strut can then be more certainly classed under pin- or round-ended columns, and calculated accordingly. But it frequently happens that the line of thrust is, from the nature

of the connections, not axial, and the loss in strength should be allowed for accordingly.

These considerations lead to the adoption of a low working resistance, or of a large factor of safety, if the strength of the strut is calculated from the usual formulæ.

The variation of stress in the bracing caused by unequal loading will, of course, have received attention in the preparation of the stress diagram.

The form of section for struts will vary with the dimensions of the principal, and the position occupied in the truss. The section may be a simple angle or tee, two tees back to back, kept apart by cast-iron distance pieces, and riveted through, two flats treated in similar manner (see Fig. 141); while a section consisting of four angles, arranged as shown in Fig. 144, and kept in position by special castings, has been used with success in some of the largest examples of the bow-string truss in this country. Tubular struts are occasionally used, but require connections at their ends of a somewhat special character. The use of cast iron for struts was frequent in roofs of old-fashioned design, but has been replaced in modern roof-work in wrought iron or steel by the types above referred to.

The ties are usually constructed as pure tension members, and may be of any of the sections previously alluded to and used for the main tie, as round or flat bars.

In some cases, however, angles are employed both for struts and ties, with riveted connections, and in this way an effective and economical truss (economical, that is, in cost of construction) is obtained. As all the bracing members are thus capable of resisting compressive stresses, the changes of sign in stress in the bracing arising from unequal loading are met.

Purlins.—The arrangement and construction of these important members of a roof structure must now be considered, and it will be found that the class of roof covering to be adopted will have considerable influence on their design.

Thus, if a covering of slates or tiles be used without boarding or battens, the purlins will take the form of angle laths, spaced at a distance corresponding with the gauge of the slates or tiles, which are wired to them. This is a common form of covering in gas-retort houses or similar structures.

Again, if slates, zinc, lead, or copper are laid on boarding, then the distance apart of the purlins will be regulated by the maximum

span, which can be assigned to the boarding, allowing for dead and live loads, and with a proper amount of stiffness, or absence of undue deflection. Or if zinc be laid with Italian corrugations upon wood rafters, then the maximum span allowable for the rafters will determine the pitch of the purlins; while if one or other of the numerous patent forms of glazing be adopted, it will be found necessary to accommodate the spacing of the purlins to the details of the system employed, including also consideration of the maximum length of sheet-glass to be used, and the allowable span of the sash-bars.

A similar condition will be found to prevail when zinc is laid on boarding with drips, and the length from drip to drip will be ruled by the standard length of zinc sheet to be used, allowance being made for the length of sheet taken up in forming the drip, tucks, overlaps, etc.

Generally, conditions such as those outlined above will govern the pitch and setting out of the purlins, and following thereon the arrangement of the bays of intermediate bracing, and the subdivision of the main rafter.

The details and sections of the purlins themselves will be dependent upon the load to be carried, and their span, that is, the distance apart of the main trusses. This latter dimension usually varies with the span of the roof, although in many cases other considerations may govern the distance apart of main principals, such as the distance apart of column foundations (when ruled by local circumstances), the arrangement of the piers in supporting walls, and the like. Thus, for spans up to, say, 40 feet, a very usual distance centres of principals is from 6 to 10 feet. Principals of spans of 100 to 200 feet are commonly spaced 25 to 35 feet apart, while trusses of such exceptional spans as 300 feet or upwards may be from 50 to 70 feet apart.

The purlins in structures of such dimensions as the latter are lattice girders, of considerable depth and weight; those of, say, 25 to 35 feet in span may consist of trussed angles or tees, or occasionally rolled joists, while those of 6 to 10 feet span are usually either single angles, tees, channels, or rolled joists of light section.

Where the main principals or trusses are as much as 25 feet apart, intermediate rafters resting upon the purlins are frequently adopted, thus subdividing the spaces to be covered, and resulting in a span which can be met by one or other of the roof coverings above referred to.

Purlins of considerable span, such as 25 feet, when consisting of braced beams or lattice girders, and arranged so that the plane of the web of the girder is normal to the slope of the main rafter, which is frequently the case, are subject to a twisting moment, due to their centre of gravity having a lever arm about the point of support, which should not be overlooked.

This consideration points to the desirability of so arranging for heavy purlins that their webs lie in a vertical plane, and this will usually lead to the adoption of vertical members in the system of intermediate bracing in the main truss.

Such vertical members are also useful in the case of hipped roofs, and simplify attachments.

Such considerations will, however, only apply in the case of purlins of considerable span and weight.

Roofing Accessories—The collection and disposal of Rain-water or Melted Snow—Skylights and Ventilators.—The arrangement of the general scheme of roof drainage, and of the principal and secondary gutters, with their cesspools and downpipes or spouts, should always receive the very careful attention of the designer.

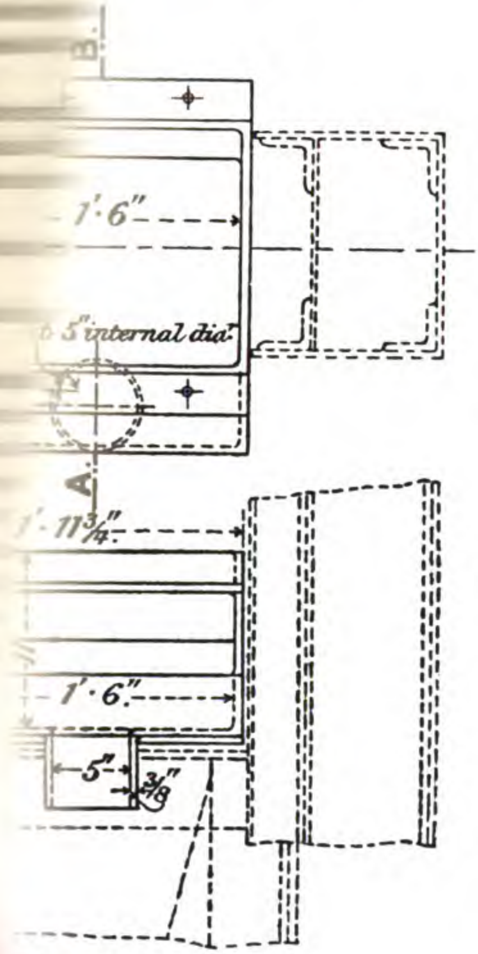
The material used most frequently for roof guttering and rain-water pipes in iron constructions is cast iron, although occasionally riveted steel gutters are used in special situations. In timber roofing for ordinary building construction, timber guttering, lined with lead or zinc, is commonly used, but to this latter form of construction further attention need not here be given.[1]

The disposal of rain-water, and the necessary dimensions to be given to gutters and down-spouts for the safe drainage of any given area of roof, constitute an important branch of the science of applied hydraulics, combined with due consideration of the meteorological conditions of the site as regards the maximum rainfall to be provided for, especially if the roof is to be designed for tropical climates, where excessive rainfalls, occurring over greater or lesser periods of time, are to be anticipated, and must be duly and efficiently met, with a sufficient margin of safety against overflow and flooding.

The purely hydraulic questions connected with the flow of water in channels, and through orifices which have to work under the widely varying conditions met with in practice, cannot here be dealt with, and it is probable that much of the design of this important branch of roof construction has been carried on by empirical, or more or less rule-of-thumb, methods, based, no

[1] See "Notes on Building Construction," vol. i.

[illegible] of gutter, with its cesspool, [illegible] in Figs. 246 to 249.



ELEVATION.

[illegible]. 248, 249.

[illegible] inch = 1 foot.

[illegible], and Fig. 247 a section on CD in

[illegible] outlet and down[illegible] d
[illegible]der to clear the [illegible] .

doubt, on practical experience, but with which the ordinarily accepted rules and formulæ of applied hydraulics have had little to do.

Experimental evidence on many points connected with the design of guttering, cesspools, and drainpipes is, so far as the writer is aware, to some extent lacking, and it is much to be desired that correct information on these subjects should be extended.

The figures detailed in Table 35, and showing the average results of a considerable number of careful experiments on the rate of discharge of guttering, cesspools, and downpipes of a certain type of design, are therefore presented as a small contribution to the

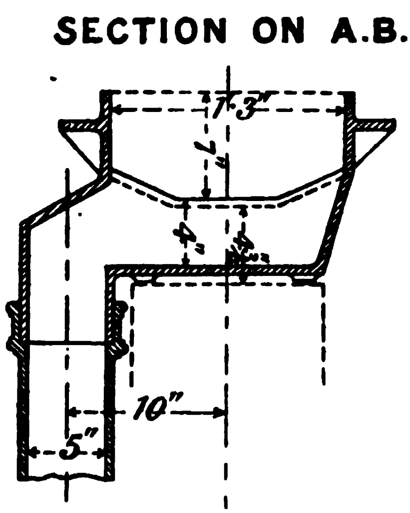


FIG. 246.
Scale 1 inch = 1 foot.

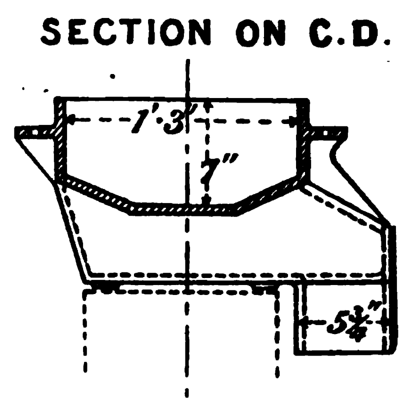


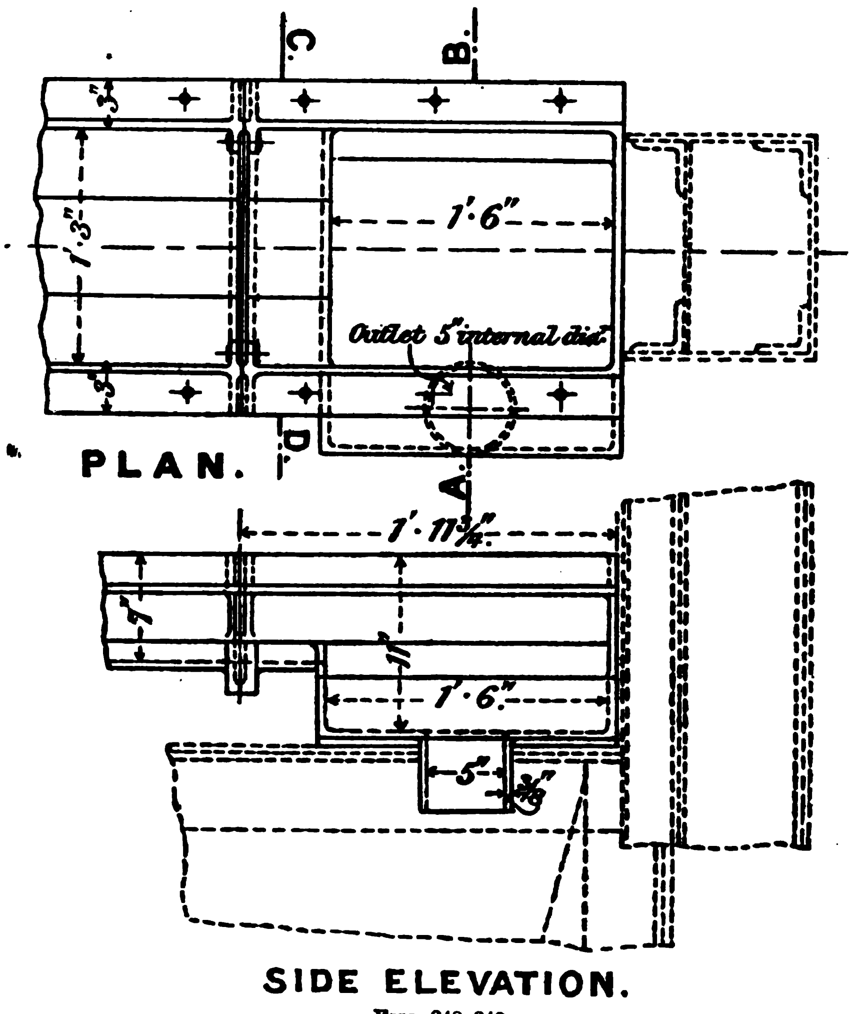
FIG. 247.
Scale 1 inch = 1 foot.

general subject, and are, of course, applicable only to the precise details described. They may serve perhaps to bring out some of the conditions with respect to flow and discharge which are to be met with in practice.

The gutter experimented upon is shown in Fig. 247, the flanged joints being partly internal and partly external, as shown in Fig. 248.

A similar section of gutter is shown in Fig. 250 with flanges wholly on the outside, as shown in Fig. 251. These flanges are machined, bolted, and made watertight with rust cement, the section of joint being of similar type to that in Fig. 257.

The connection of the normal run of gutter, with its cesspool, which occurs at a stopped end, is shown in Figs. 246 to 249.



Figs. 248, 249.
Scale 1 inch = 1 foot.

Fig. 246 is a section on AB, and Fig. 247 a section on CD in Fig. 248.

It will be observed that the outlet and downpipe are arranged out of centre of the gutter, in order to clear the supporting lattice

girder below, and the detail illustrates one of those practical conditions in the design of gutterwork to which purely hydraulic considerations have occasionally in some degree to give way. Fig. 248 is a plan, and Fig. 249 an elevation of the cesspool.

The actual section of valley gutter, with its cesspool and downpipe forming one complete bay of roof drainage, was tested in position in the roof with the results detailed in the following table, the gutter being filled successively to the depths shown, and the contents allowed to discharge themselves freely through the apertures shown in the figures by the removal of suitably arranged plugs or valves.

TABLE No. 35.

TABLE SHOWING THE RESULTS OF EXPERIMENTS TO ASCERTAIN THE RATE OF DISCHARGE OF RAIN-WATER FROM THE GUTTER, CESSPOOL, AND DOWNPIPE SHOWN IN FIGS. 246 TO 249.

Internal diameter of downpipe in inches.	Fall in level of water in gutter.	Contents discharged in cubic inches.	Observed mean duration of flow in seconds.	Discharge in cubic inches per second.
5	6″ to 5″	18,975	11·5	1650·0
5	5″ to 4″	18,975	26·5	716·0
5	4″ to 3″	18,975	37·0	512·8
5	3″ to 2″	18,975	66·5	285·3
5	6″ to 2″	75,900	141·5	536·4

The vertical length of downpipe attached to the cesspool in the above experiments was about 31 feet 6 inches to the junction with the rain-water drain, and was 5 inches internal diameter throughout. The necessities of design in arranging for the reception and attachment of the downpipe to a column of lattice construction, and in the passing of roof and traveller girders, gave rise to about 5 bends of 45 degrees each, so that the discharge was subject to conditions not more favourable than those usually found in practice.

The rapid decrease in discharge with the decrease of head over the mouth of the cesspool and downpipe will be observed, and the rate of decrease is greater than that due to theoretic loss of velocity following on loss of head. Probably the special conditions induced

by the shape and dimensions of cesspool, the ratio of the area of the downpipe to the area of cesspool; and the inability of the volume and height of water contained in the cesspool over the mouth of the downpipe to maintain the latter in the condition of

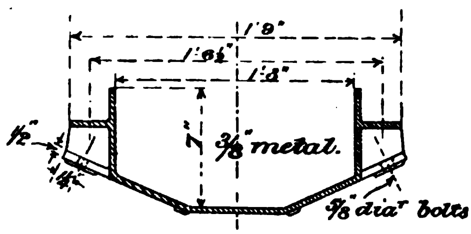


FIG. 250.
Scale 1 inch = 1 foot.

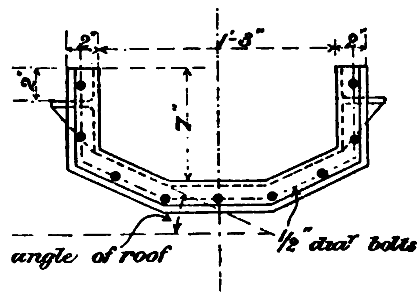


FIG. 251.
Scale 1 inch = 1 foot.

"full flow," are sufficient to account for the comparatively small discharge at low heads.

But these are conditions common in greater or less degree to most details of guttering and rain-water disposal, hence the value of practical experiment in cases such as the above—and a wide field is open for the student in the carrying out and analysis of

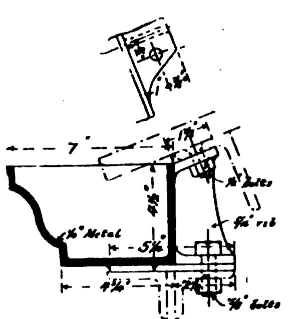


FIG. 252.
Scale 1½ inch = 1 foot.

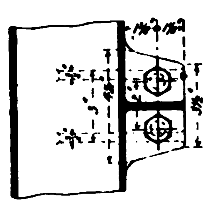


FIG. 253.
Scale 1½ inch = 1 foot.

experiments similar in kind, but covering a wider range of investigation into the influence of cross-section of gutter, the best form of cesspool, and the precise value of discharging power of varying diameters of downpipe.

The arrangement indicated in the figures and experimented upon as described above, was intended for use in buildings of large area, with numerous valley gutters, and if an attempt were

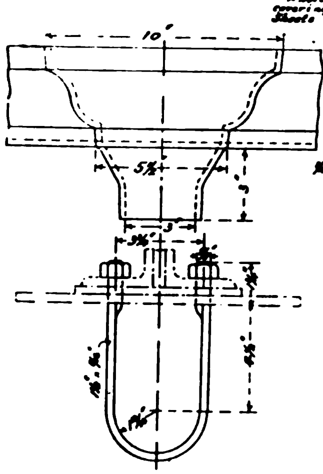


Fig. 254.
Scale 1¼ inch = 1 foot.

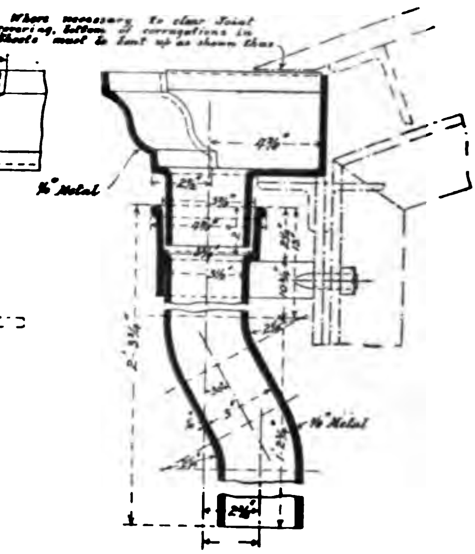


Fig. 255.
Scale 1¼ inch = 1 foot.

made to deduce from the above figures the maximum area of roof surface which could safely be drained by such an arrangement of gutter, cesspool, and downpipe, the calculation would include

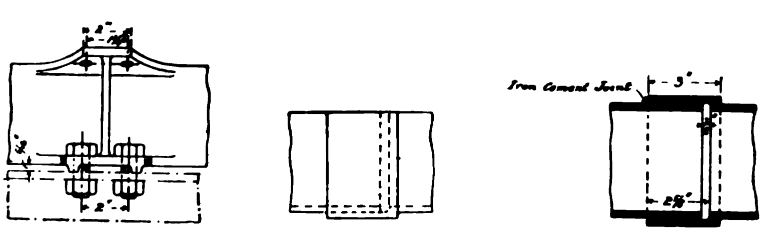


Fig. 256.
Scale 1¼ inch = 1 foot.

the following considerations. Assuming that no accidental obstruction occurs in the gutter or downpipe, such as collections of leaves, mud, etc., it will be desirable that the surface of water

in the gutter shall never, during the period of heaviest rainfall, be allowed to stand higher than will give a certain margin of safety

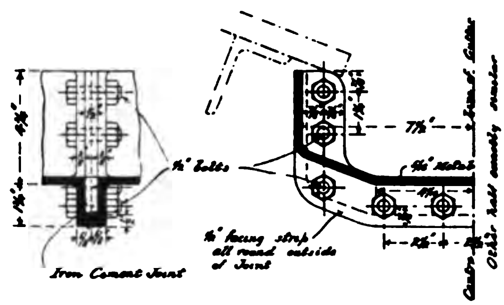


FIG. 257.
Scale 1¼ inch = 1 foot.

to prevent overflow and flooding, especially in those cases where such an occurrence would be attended with serious annoyance and

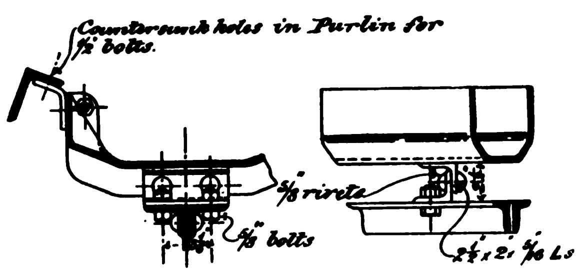


FIG. 258.
Scale 1 inch = 1 foot.

discomfort. In estimating this margin, it will be necessary to remember that the surface of water in the gutter flowing towards

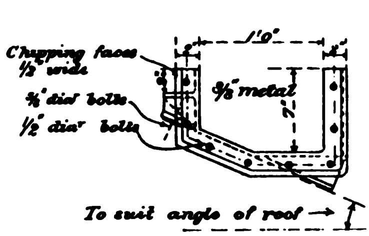


FIG. 259.
Scale 1¾ inch = 1 foot.

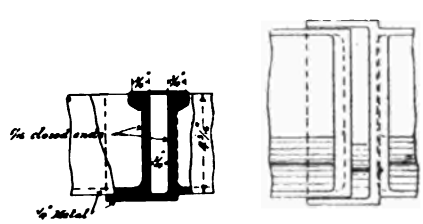


FIG. 260.
Scale 1¼ inch = 1 foot.

the cesspool or outlet will not be level, but will assume an hydraulic gradient. In the experiments above described, and with the section of gutter shown in the figures, this gradient was found to be, in a length of about 106 feet, in the ratio of about 1 in 600.

Allowing, then, for this gradient, and determining the desirable amount of margin below the lip of the gutter at the highest end of the flow, we can deduce the approximate head which will be

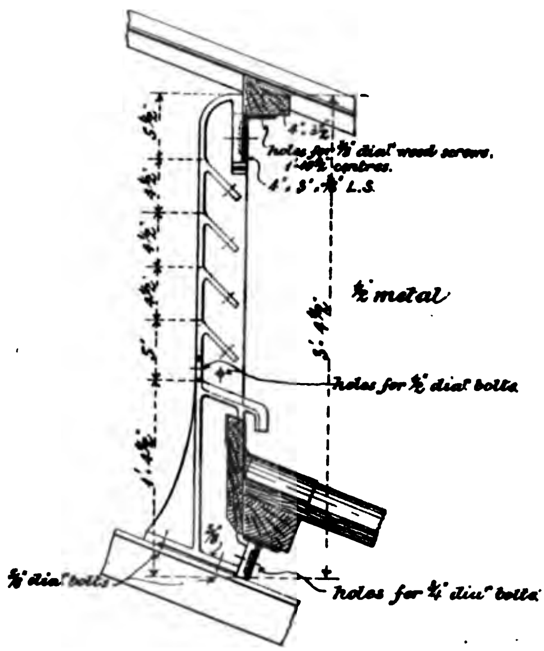


Fig. 261.
Scale ¾ inch = 1 foot.

available at the cesspool, and the corresponding discharge, which, as we have seen, will rapidly decrease with a diminishing head. The amount of discharge which can be relied on under these conditions can then be equated with the maximum rainfall per hour, or some still shorter period, and the area of roof which can be safely drained under the conditions assumed can thus be ascertained.

In the example quoted the actual area of roof drained was

about 4945 square feet to one 5-inch downpipe, and it is calculated that, assuming a maximum and exceptional storm rainfall of about $2\frac{1}{2}$ inches per hour, the highest level of water in the gutter would be about 3 inches below the edge, giving a margin of safety of 3 inches before the gutter brimmed over, thereby affording a possible increase of head under emergencies which would, from the experiments, be attended by a rapid increase in rate of discharge.

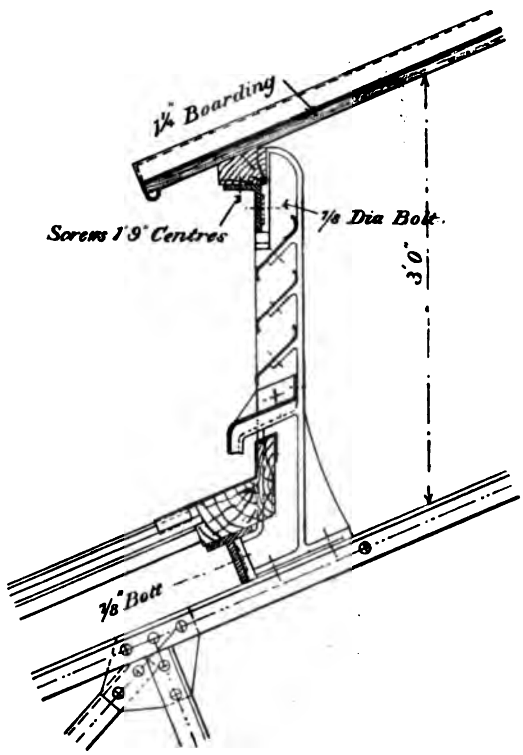


Fig. 262.
Scale ⅜ inch = 1 foot.

The above remarks apply to gutters of a constant section laid, as they frequently are, for simplicity of construction in details, dead level from end to cesspool. Gutters laid with drips and of varying cross-section fall under a different category.

Various forms of gutter with their attachments, both valley, eaves, and wall gutters, are illustrated in the figures which follow, but no attempt has been made in these notes to illustrate the

numerous types of ornamental cast-iron guttering for architectural purposes which are to be found elaborately illustrated in the catalogues of art ironfounders, and many of which exhibit much beauty of design. The examples here given are purely utilitarian, and are such as would be found in stores, warehouses, station-roofs, and the like.

An eaves or fascia gutter with details of attachments and outlets

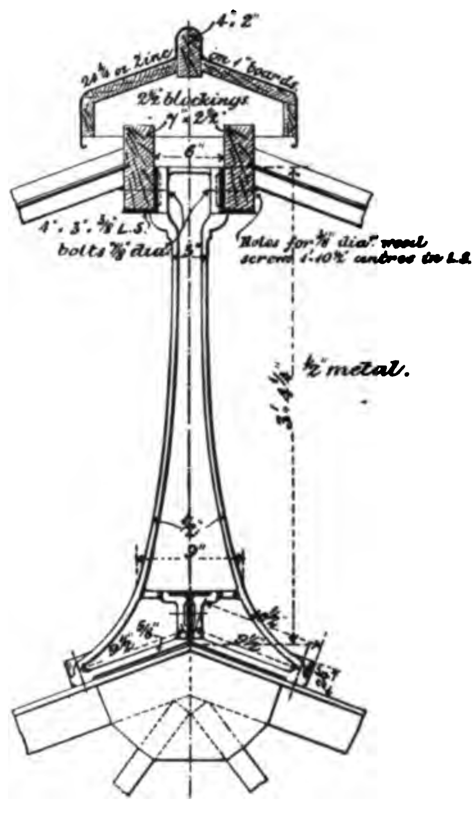


Fig. 263.
Scale ¾ inch = 1 foot.

is shown in Figs. 252 to 256 inclusive. Another type of valley gutter is shown in Figs. 251 and 258. A wall gutter is shown in Figs. 259 and 274, while in Fig. 272 is shown a wall gutter associated with a cast-iron tank forming portion of a roof covering, and further alluded to in Chapter III., p. 172.

In long straight stretches of guttering, expansion joints should be introduced, and to avoid the difficulty of making such a joint thoroughly watertight, the expedient is frequently adopted of making the necessary provision for expansion by leaving a space between two stopped ends of gutter as shown in Fig. 260, and roofing over the space so left either with a lead capping as in Fig. 260, or by a cast-iron capping or saddle piece.

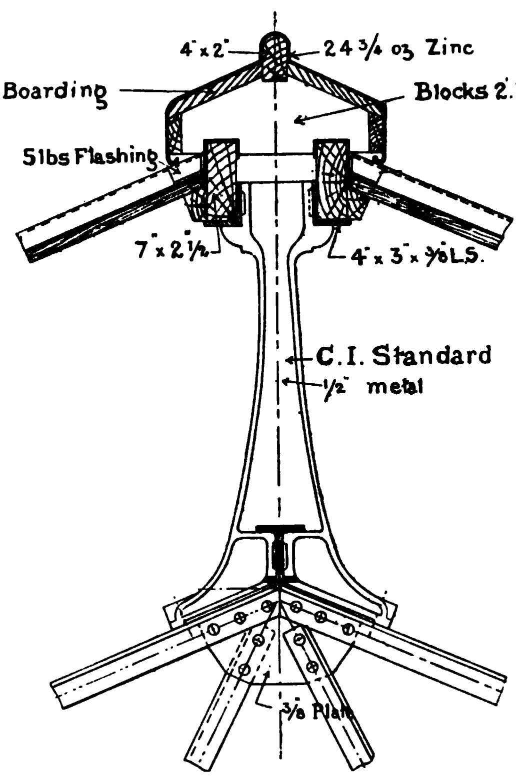


FIG. 264.
Scale ⅜ inch = 1 foot.

Such stopped ends must be considered as the summit levels, or parting of the waters, in the general system of drainage, and the down-spouts must be arranged accordingly.

Lanterns, Skylights, and Ventilators.—These are of very various types of construction, and may be used either for lighting or ventilation, or both combined. When used for lighting, the sashes

may be arranged on the sides of the lanterns either as fixed sashes, or made to open on hinges or centres; or, on the other hand, the roof of the lantern may itself be glazed, forming a fixed light.

Ventilating lanterns may be constructed either with fixed

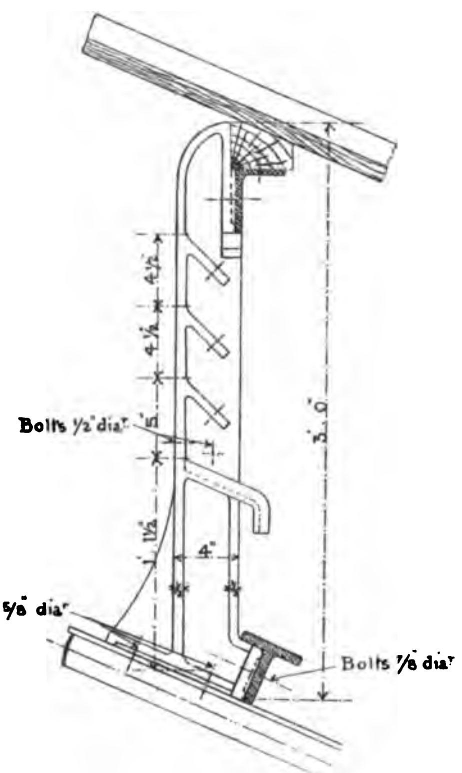


FIG. 265.
Scale 1 inch = 1 foot.

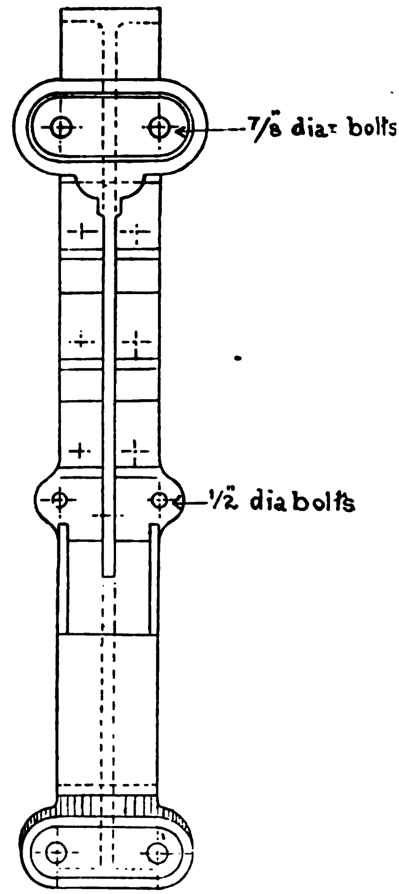


FIG. 266.
Scale 1¼ inch = 1 foot.

louvre blades, or with an arrangement of blades made to turn upon a centre, with a common gearing, enabling the ventilation openings to be opened or closed, or placed partially open at discretion. Glazed sashes or solid panels hung on centres will also effect the same purpose of ventilation, while in lanterns with

fixed louvres at the sides, an additional subsidiary ridge ventilator, with fixed opening, is frequently added.

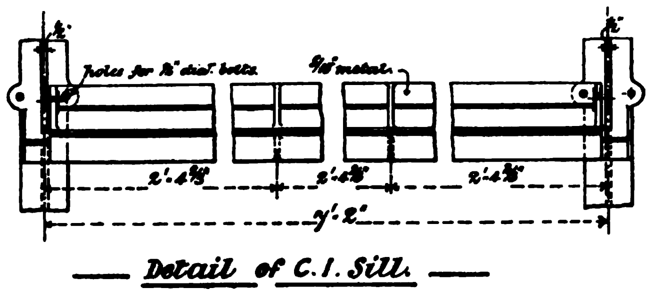


Fig. 267.
Scale ¾ inch = 1 foot.

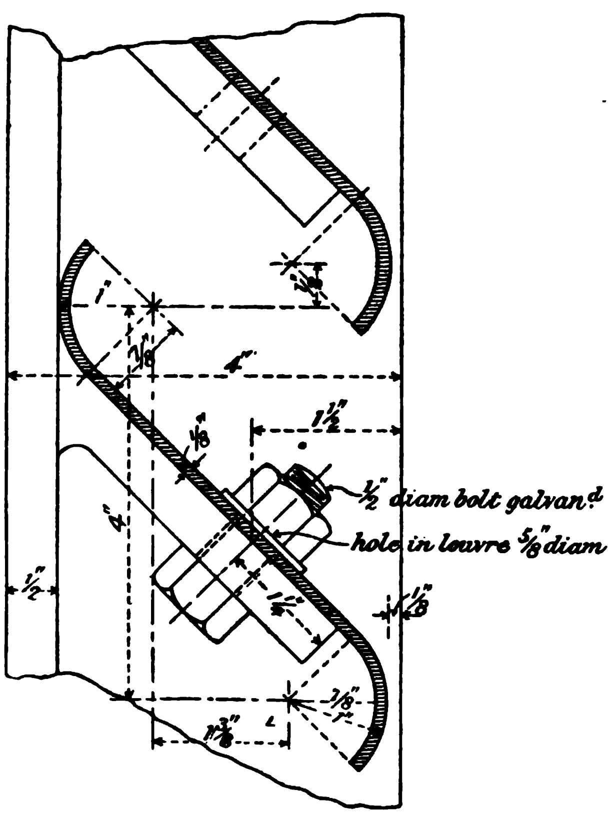


Fig. 268.
Scale half full size.

In buildings of a superior class, a type of ventilation is required which shall be reliably weathertight, and this condition is usually met by the construction of sashes in superior joiner's work, with weather bars, grooved sills, or other devices for the proper exclusion of wind and water.

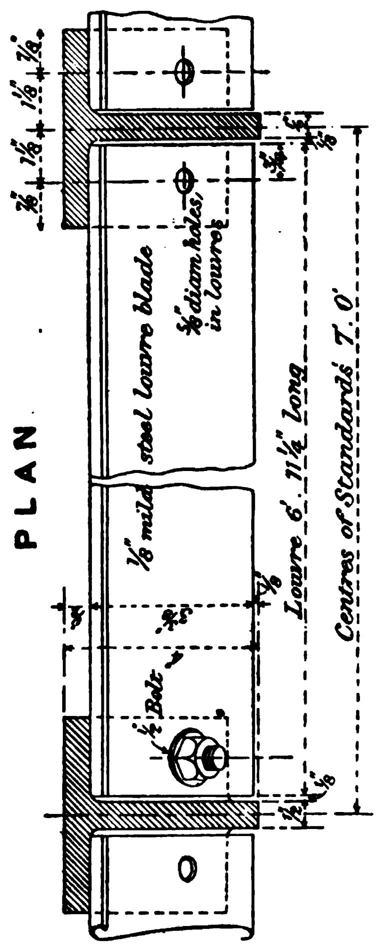


Fig. 269.
Scale 3 inches = 1 foot.

These forms of construction cannot here be further alluded to, but some attention may conveniently be given to a class of ventilating lantern frequently met with in the roofs of machine shops, boiler shops, engine houses, factories, and the like, where ventilation is essential, and where a form of construction may be

adopted which, if not absolutely weatherproof, is admissible under the circumstances.

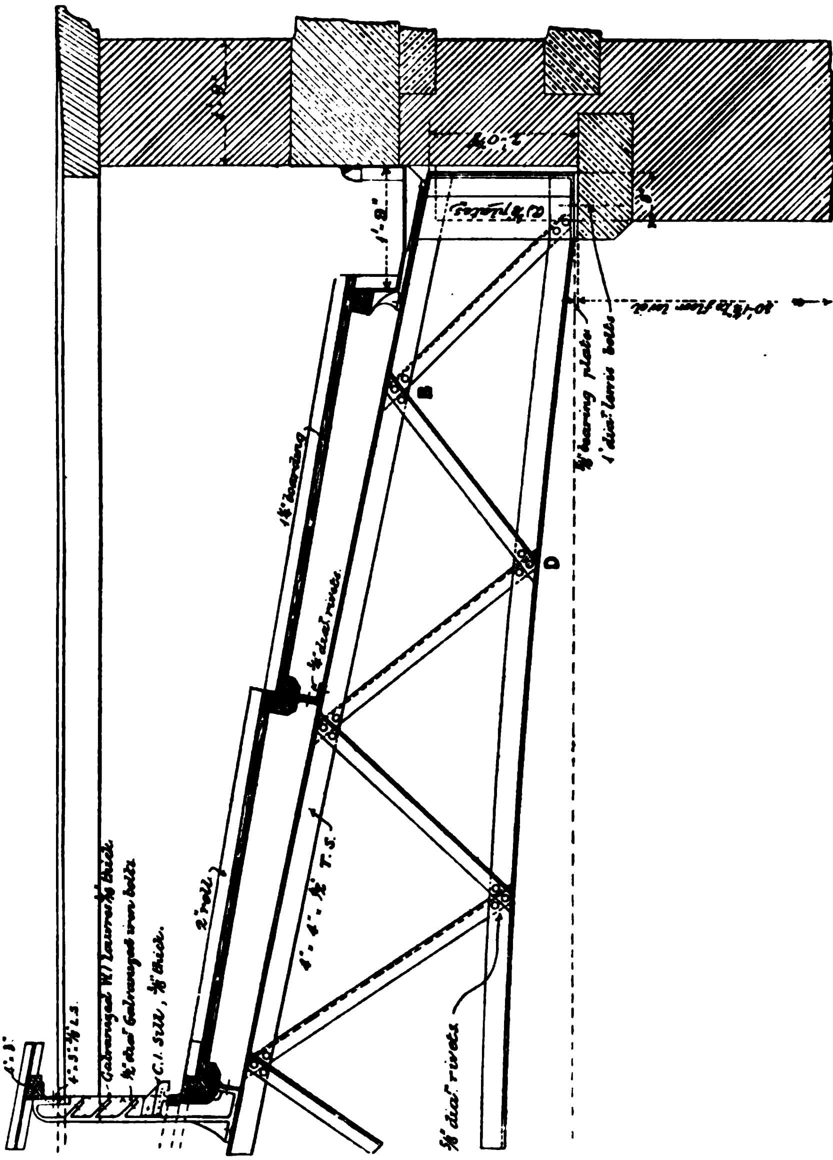


FIG. 270.
Scale ⅜ inch = 1 foot.

Ventilating lanterns of this type are shown in Figs. 271 and 281, of the same form of construction but of different pitches.

Both lanterns have fixed louvres at the sides, a glazed roof, and a subsidiary ridge ventilator.

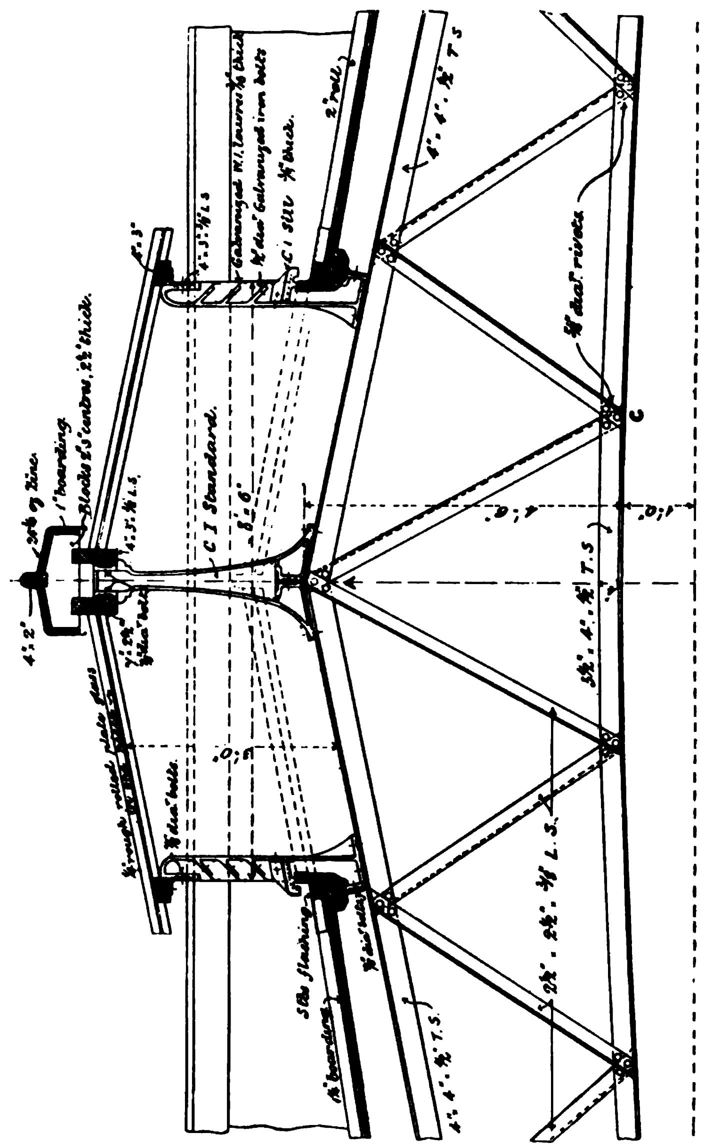


FIG. 271.
Scale [illegible] inch = 1 foot.

The structure of the lantern consists of two sides and one centre standard of cast iron, the side standards being arranged to receive

three or four rows of galvanized mild steel or wrought iron louvre blades, together with a special cast-iron sill piece forming the lowermost member of the louvre system, and throwing off the drippings of the louvre blades on to the main roof covering below,

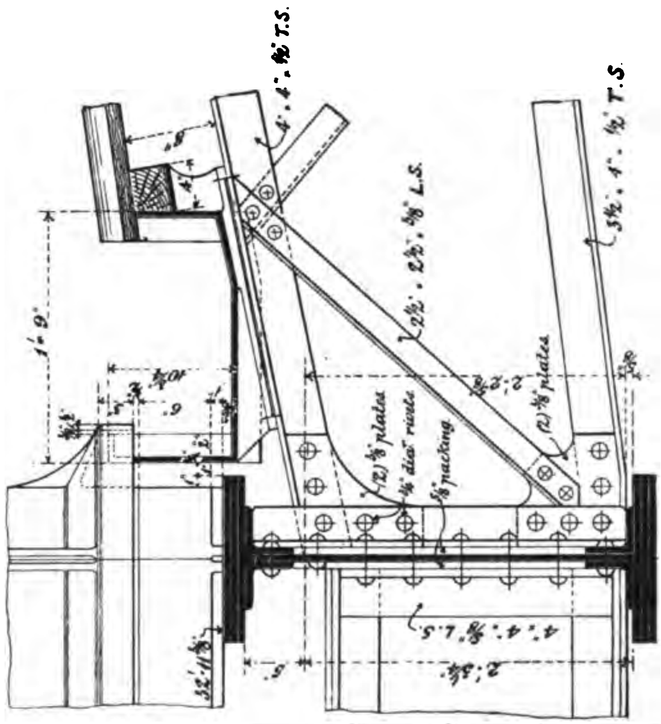


FIG. 272.
Scale ¾ inch = 1 foot.

FIG. 273.
Scale ¾ inch = 1 foot.

which in the figures shown is of zinc on boarding and rolls. The ridge ventilator is framed in timber as shown, and covered with zinc.

Side elevations of the side standards are shown in Figs. 261 (four blades), and also in Fig. 262 (three blades). Side elevations

of the centre standard are shown in Figs. 263, 264; the former showing details of glazed lantern roof with wood sash bars; the latter, details of a lantern roof of zinc on boarding.

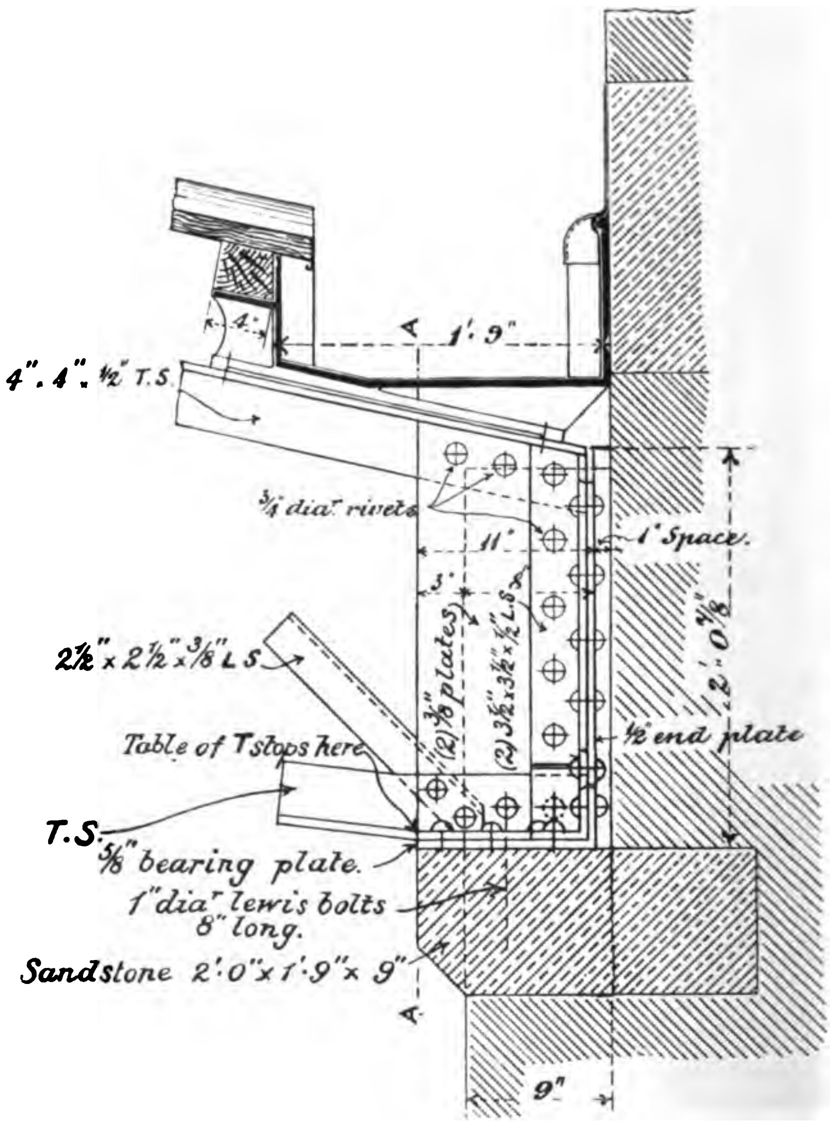
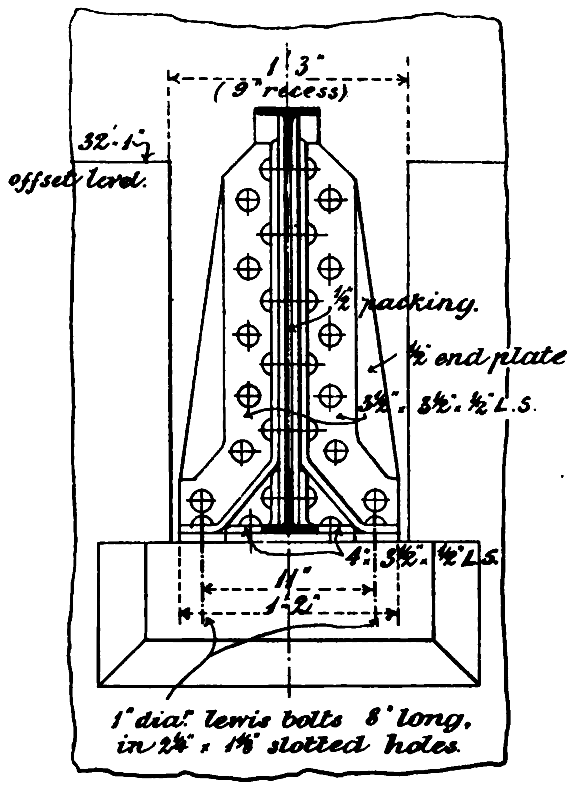


FIG. 274.
Scale 1 inch = 1 foot.

Fig. 265 is an enlarged detail of the side standard shown in Fig. 262, the blades being removed, and Fig. 266 is a front elevation, showing the provision made for the reception of the cast-iron sill piece mentioned above, and which is intended to catch any rain-water which may creep down between the ends of the louvre blades



—*Elevation on line A.A.*—

FIG. 275.
Scale 1 inch = 1 foot.

and the faces of the standard, and throw it off on to the roof covering below. Fig. 267 shows a sill piece in elevation.

The ventilating louvres above referred to are of the fixed type, incapable of being closed, and the slope, dimensions, and overlap of the louvre blades have therefore to be so arranged as to give

the greatest possible amount of watertightness, while preserving a free entrance and exit of air for ventilating purposes.

A large scale detail of the blades and their attachment is given in Figs. 268 and 269, which show the means adopted to

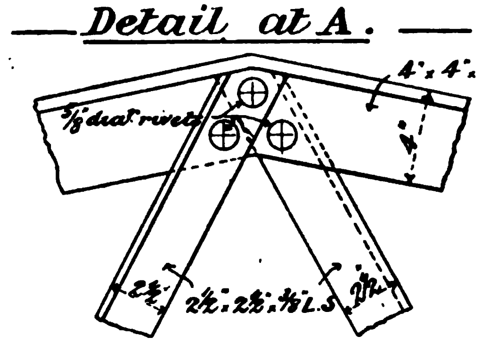


FIG. 276.
Scale 1¼ inch = 1 foot.

secure these requirements, which were successfully attained in the cases under description.

A fair idea of the degree of watertightness obtainable by such an arrangement as that shown can be obtained by the construction

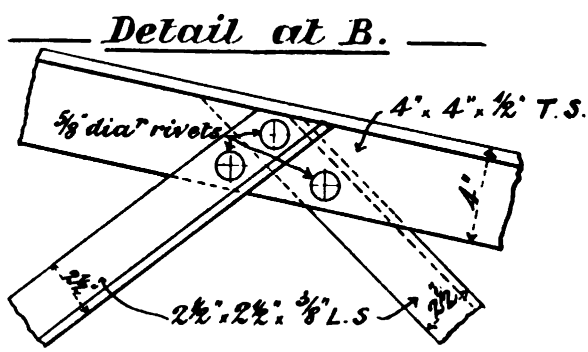


FIG. 277.
Scale 1¼ inch = 1 foot.

of a model of two or more rows of the blades in zinc full size. Water sprinkled or poured upon the blades will collect in drops at the lower edge, and if a drop be subjected to a powerful current of air, as, for example, from the nozzle of a pair of bellows, it can

be ascertained whether it is possible to blow the drop over the top of the next blade below, the experiment forming a rough approximation to the condition of rainfall, combined with a horizontal or

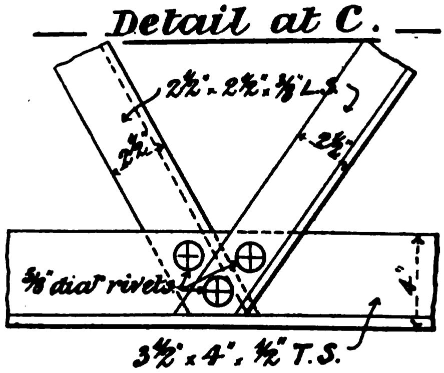


Fig. 278.
Scale 1½ inch = 1 foot.

inclined current of air in a gale of wind blowing across the lantern.

Louvre blades which warp or sag after erection present a very unsatisfactory appearance, and, in consequence the distance apart

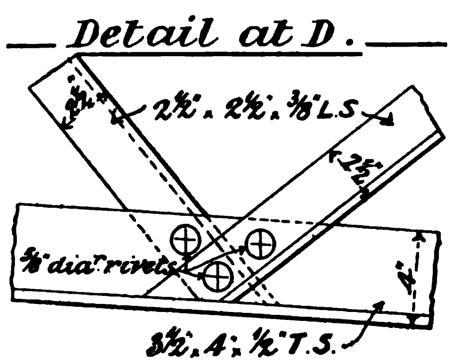


Fig. 279.
Scale 1½ inch = 1 foot.

of louvre standards should be regulated so as to give sufficient stiffness to the blade, or the cross-section of the blade must be so designed as to give the requisite stiffness for the span to be adopted. In the cases above described the blades were capable of

spanning a distance up to about 7 to 8 feet, but beyond this distance an intermediate support became desirable.

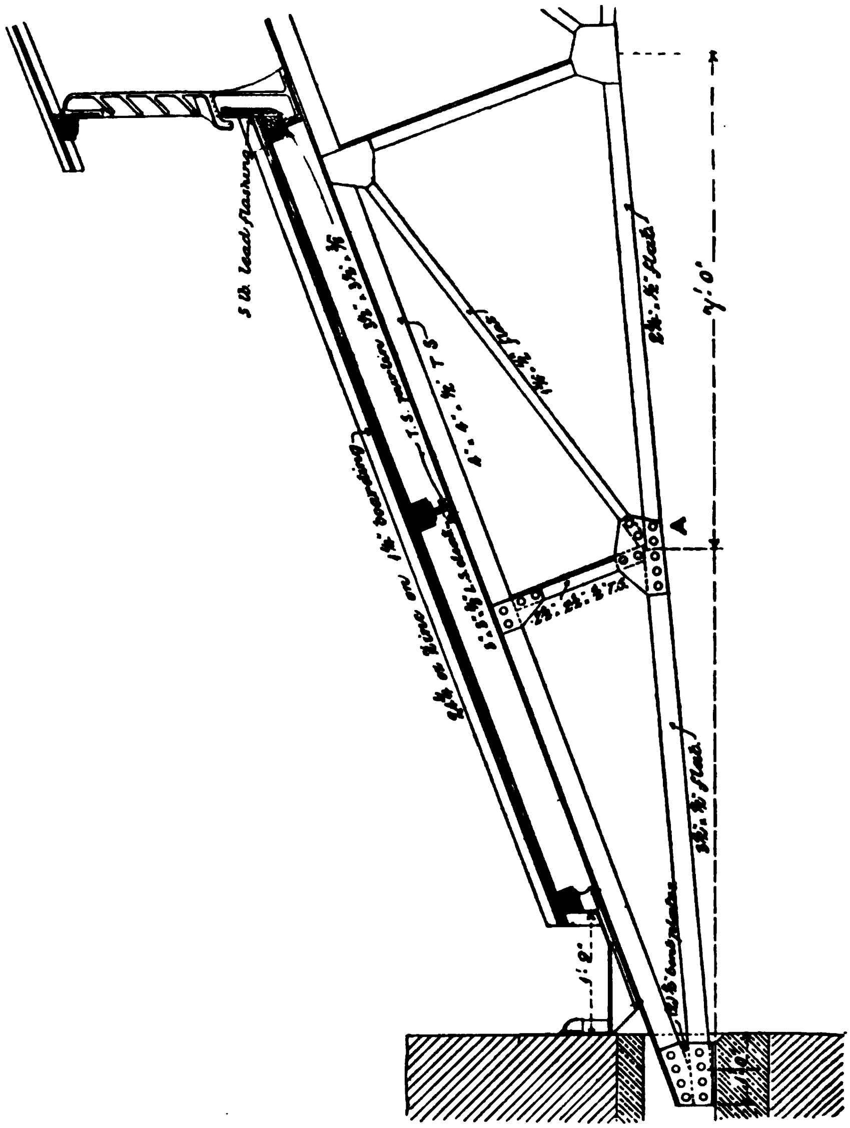


FIG. 280.
Scale 1 inch = 1 foot.

Roof lanterns and ventilators usually occupy exposed positions, and all their fastenings and connections should be such as will afford due security under these conditions.

Roofs of flat pitch are frequently adopted in cases where, for architectural reasons, it is desirable that the roof construction

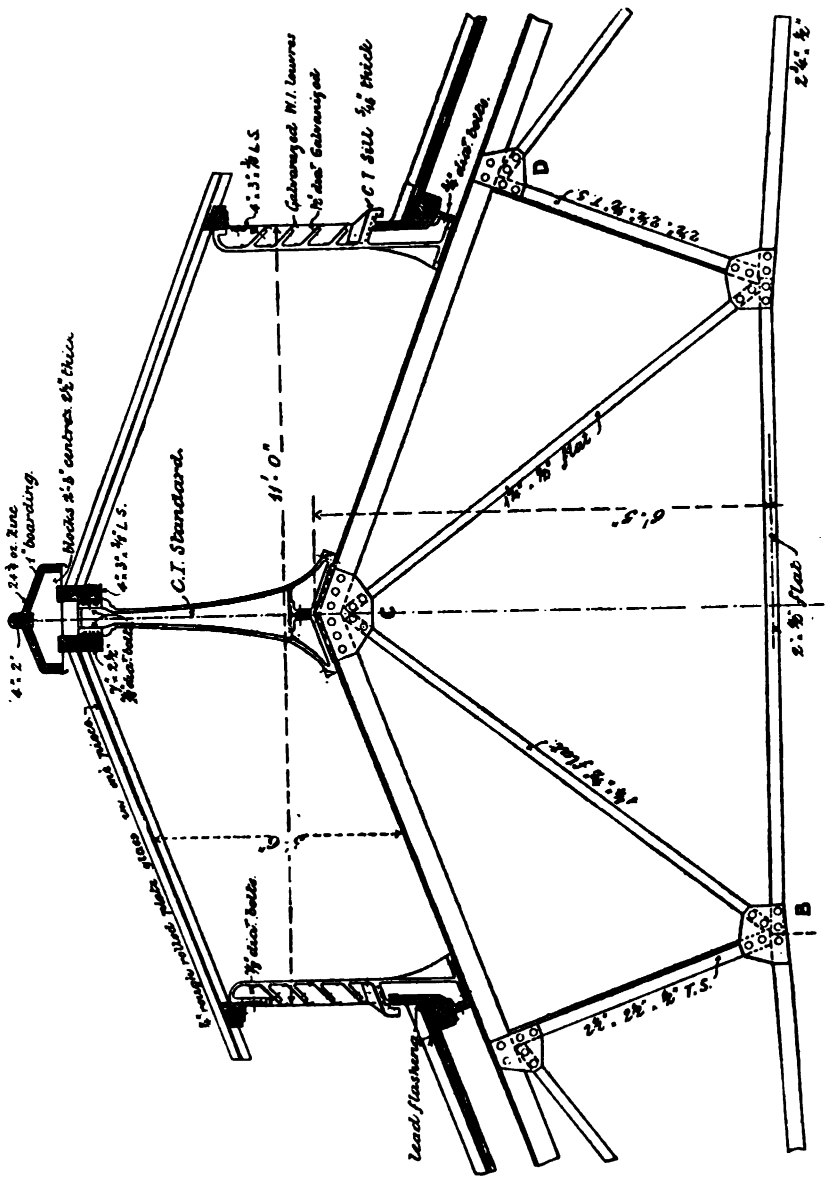


Fig. 281.
Scale ½ inch = 1 foot.

should be concealed behind parapet walls or other architectural

feature. This condition gives rise to a class of roof truss which approximates more to the form of a lattice girder, with sloped

Detail at A.

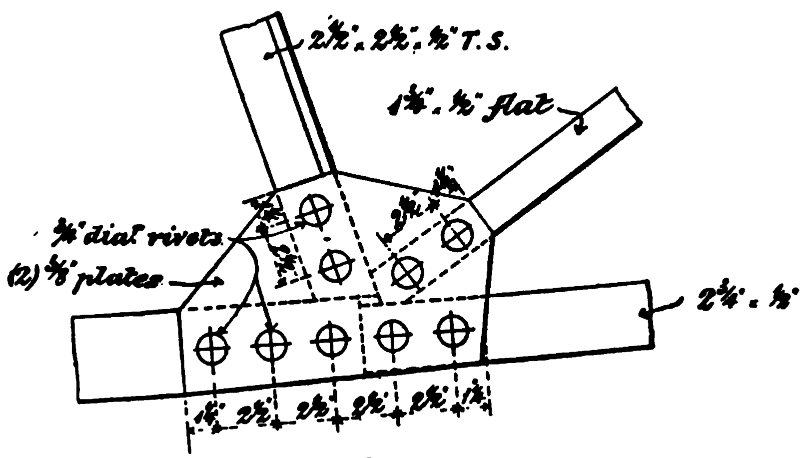


Fig. 282.
Scale 1½ inch = 1 foot.

Detail at B.

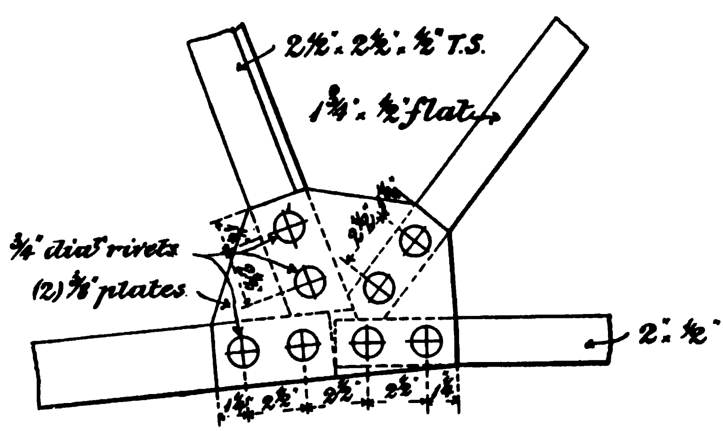


Fig. 283.
Scale 1½ inch = 1 foot.

upper flange, than to the ordinary form of roof principal usually classed under that term.

An example of this type is shown in Fig. 270, which shows a

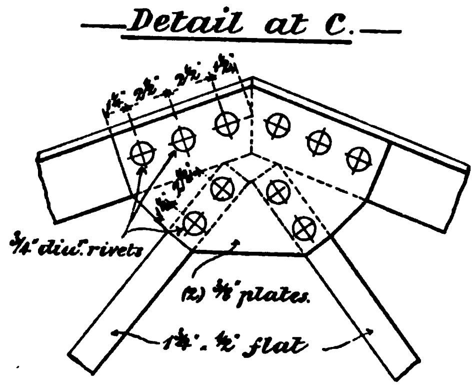


FIG. 284.
Scale 1 1/4 inch = 1 foot.

portion of the truss nearest the wall end, the section of the wall itself with the parapet being shown, and the architectural features of the cornice and string courses being broken off for convenience.

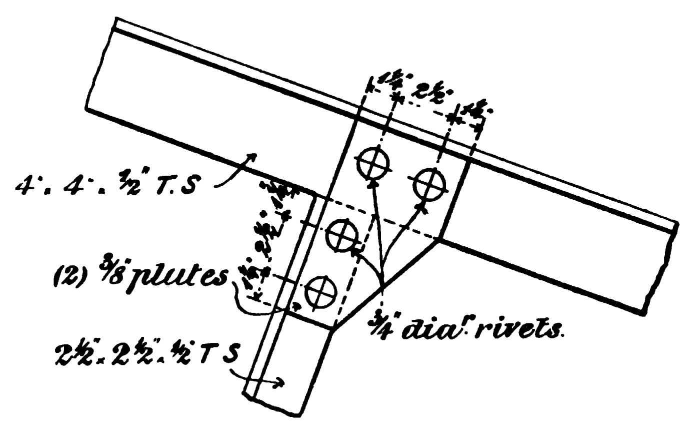


FIG. 285.
Scale 1 1/4 inch = 1 foot.

The roof is of flat pitch, the covering being of zinc on boarding laid with drips and falls as shown, and the slope of the upper member of the truss arranged to suit. The lower or tension

member is curved or given a large camber for appearance sake. In Fig. 271 the centre portion of the truss is given, showing the lantern and skylight, which is glazed with ¼-inch rough rolled

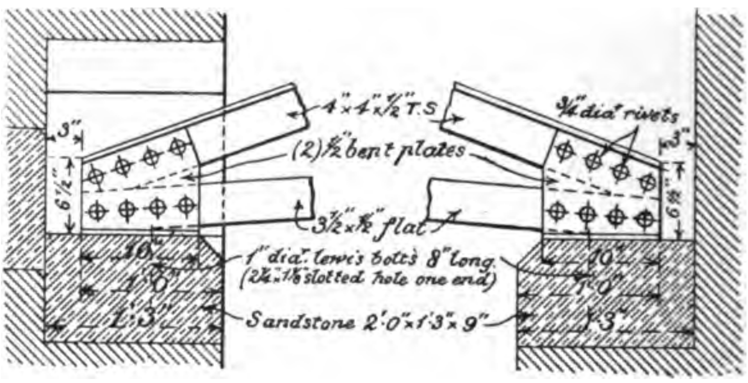


FIG. 286.
Scale ¾ inch = 1 foot.

FIG. 287.
Scale ¾ inch = 1 foot.

plate in wood sash bars, or with one of the numerous forms of patent glazing. Ventilation is secured, as in previous examples,

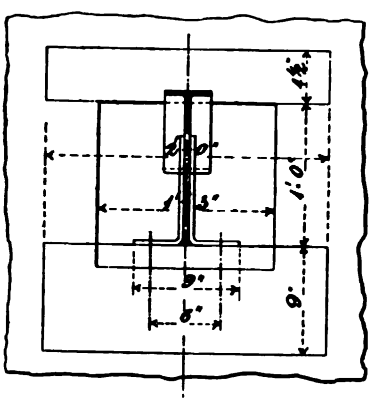


FIG. 288.
Scale ¾ inch = 1 foot.

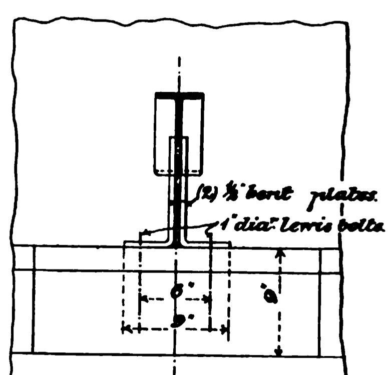


FIG. 289.
Scale ¾ inch = 1 foot.

by galvanized wrought iron or mild steel louvres secured to cast-iron standards, with an additional ventilator at the ridge of the skylight framed in timber and covered with zinc.

The end of the truss opposite the wall rests upon a riveted steel plate girder carrying a heavy water tank, and the detail of attachment to the girder, together with the assemblage at this

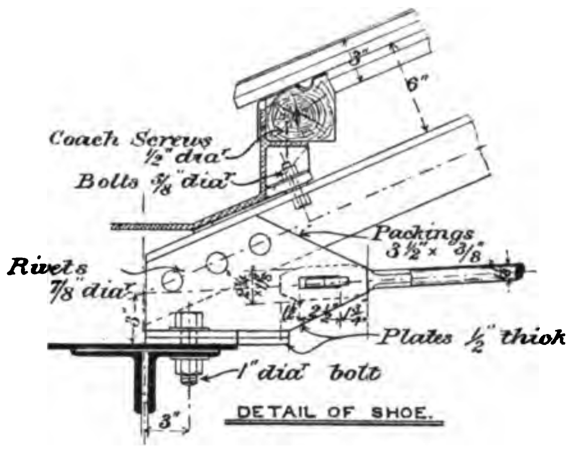


Fig. 290.
Scale 1 inch = 1 foot.

point of the truss, girder, tank, gutter, and roof-covering detail, is shown in Fig. 272, with a sectional elevation of the connection of truss to girder in Fig. 273.

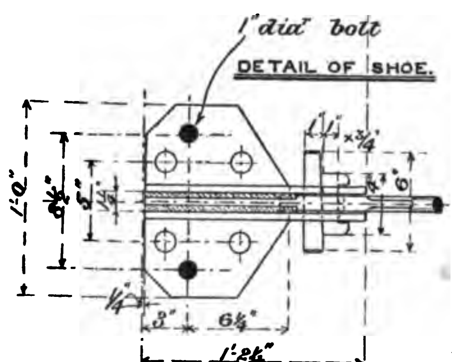


Fig. 291.
Scale 1 inch = 1 foot.

The details of the tank, together with the weathertight connection between the tank and the gutter, are referred to in Chapter III., and shown in the figures therein described.

The detail of the wall end of the truss is shown in Fig. 274, with section of the wall gutter, while a sectional elevation of the end of the truss, showing its seating on the wall, on the line AA, Fig. 274, is shown in Fig. 275.

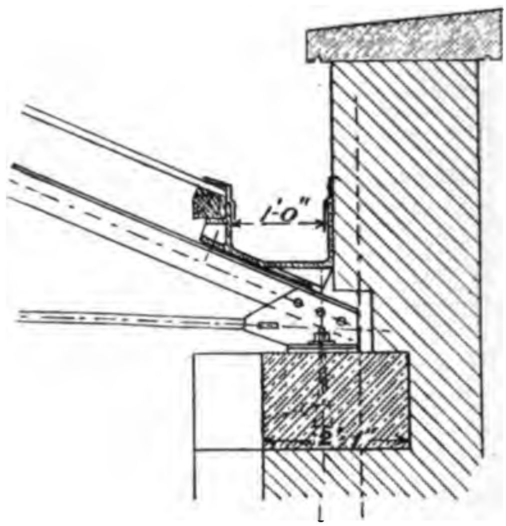


FIG. 292.
Scale ½ inch = 1 foot.

The upper and lower members of the truss are T steels, and the intermediate bracing of angles. The details of connections at

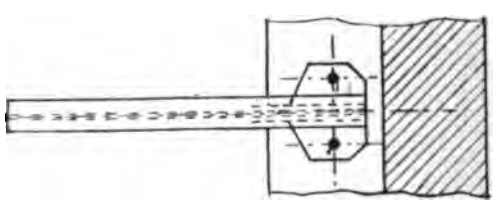


FIG. 293.
Scale ½ inch = 1 foot.

the points A, B, C, and D, Figs. 270 and 271, are shown in Figs. 276, 277, 278, and 279 respectively.

It will be observed that no joint is shown in the upper member at A, or in the lower member; this course being only permissible

where the span of the truss, or considerations of transport or erection, will allow of the truss being sent away and erected in one piece.

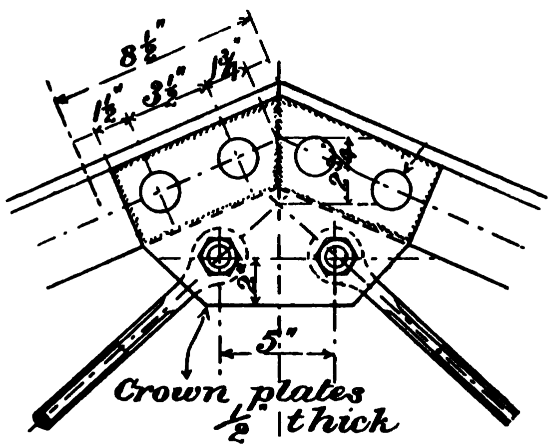


Fig. 294.
Scale 1½ inch = 1 foot.

Details of a roof principal of ordinary pitch, with a covering of zinc on boarding, and resting upon walls at both ends, are given in Figs. 280 to 289 inclusive.

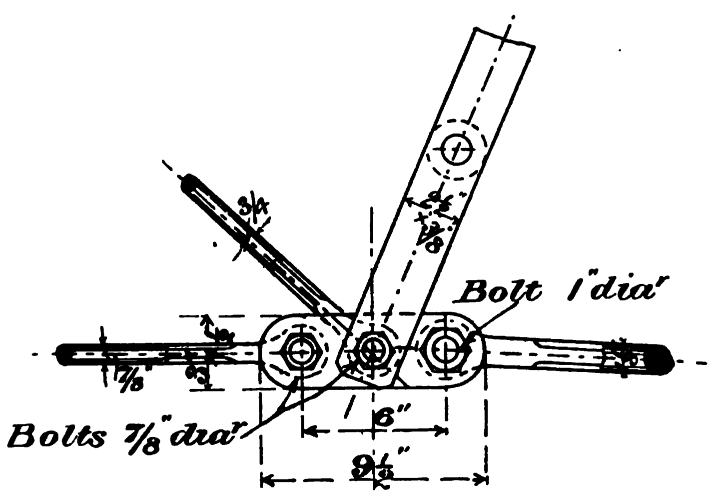


Fig. 295.
Scale 1½ inch = 1 foot.

The portion of the principal next the wall is shown in Fig. 280, while the central portion of the truss, with its lantern and skylight, is shown in Fig. 281.

This principal is composed of a tee steel top or compression member and flat bar tie or tension member, while the bracing consists of tee steel struts and flat bar diagonals.

The details of connections of these members are shown in Figs. 282, 283, 284, and 285.

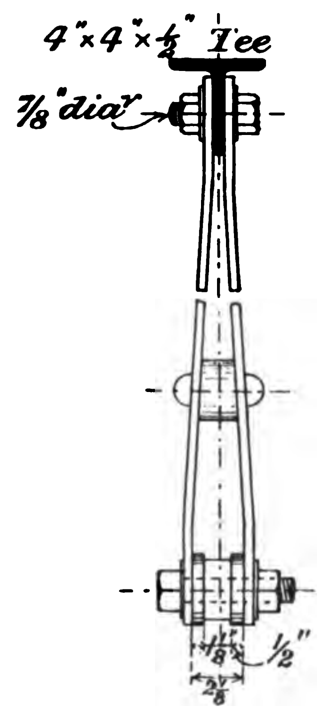


FIG. 296.
Scale 1½ inch = 1 foot.

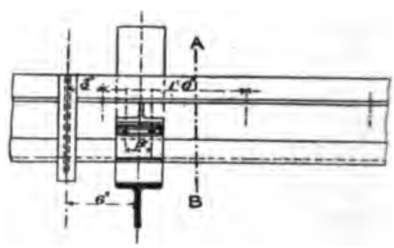


FIG. 297.
Scale ¾ inch = 1 foot.

The end shoes (in this example of a very simple type) and their seatings upon the walls at either end are shown in Figs. 286 and 287, and also in elevation in Figs. 288 and 289.

The skylight and ventilator are of a type similar to other examples described in this chapter.

The central cast-iron standard is shown in elevation in Fig. 263, and the side standard, carrying the galvanized wrought-iron louvre

blades in Fig. 261, the detail of the cast-iron sill forming the lowermost member of the ventilator, and which forms a drip over the

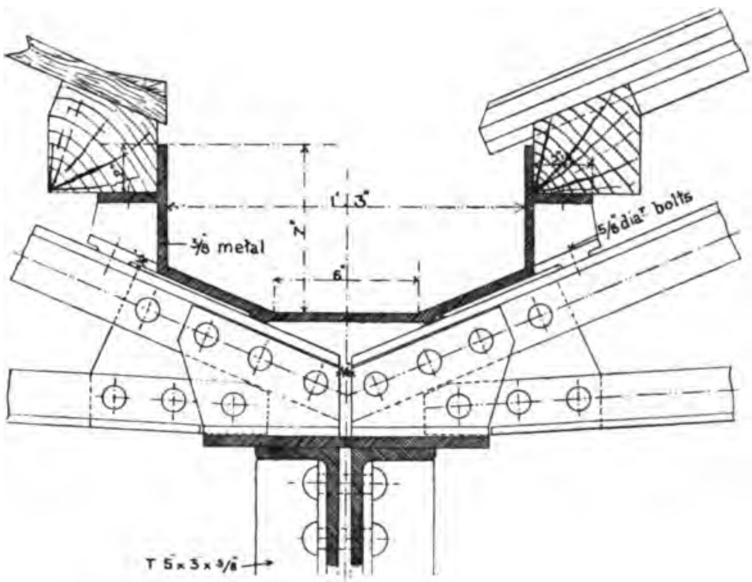


Fig. 298.
Scale 1½ inch = 1 foot.

junction of the zinc roof covering at this point, being shown in Fig. 267.

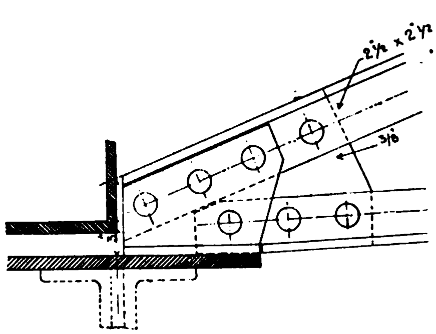


Fig. 299.
Scale 1½ inch = 1 foot.

In contradistinction to a principal with flat bar main tie, above described, we may now consider an example of the older fashioned

type of truss with round rod ties. The variation in modern practice from this form of tie has been already discussed in the previous paragraphs of this chapter.

In Fig. 290 we have the side elevation of the shoe of a roof

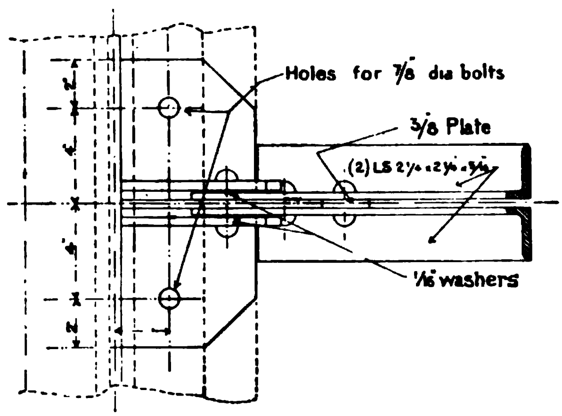


FIG. 300.
Scale 1½ inch = 1 foot.

principal of this type, with round rod tie and keyed and cottered connections. The shoe in this case rests upon the upper flange of a lattice girder, and the plan is shown in Fig. 291, which also

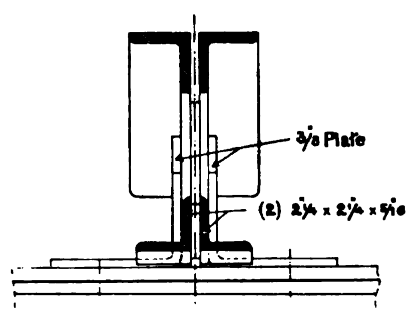


FIG. 301.
Scale 1½ inch = 1 foot.

gives further details of the key and cotter. The arrangement at the wall end of the same principal is shown in Fig. 292.

The bracing ties being also of round rod section, the details of connections become modified accordingly, as compared with a

riveted flat bar detail, and are shown in Figs. 294 and 295; the strut, being of a form frequently used in small spans of this type,

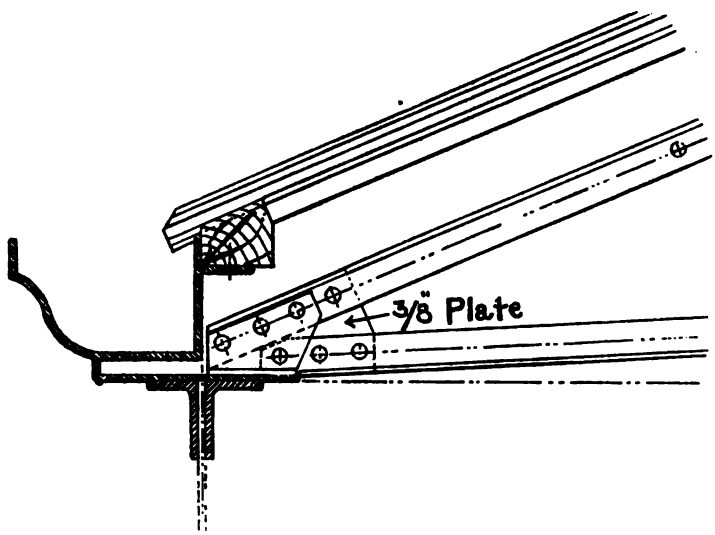


Fig. 302.
Scale 1 inch = 1 foot.

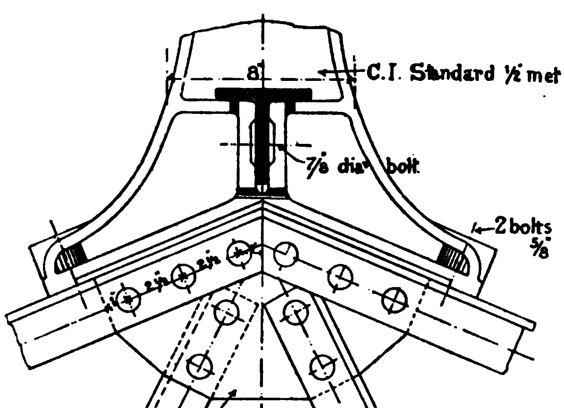


Fig. 303.
Scale 1¼ inch = 1 foot.

and consisting of a pair of flat bars kept apart by cast-iron distance pieces riveted through, is shown in elevation in Fig. 296.

The normal section of valley gutter at AB, Fig. 297, partly indicated in Fig. 290, is more fully shown in Fig. 250. The flanged connection, made in rust cement and bolted, is shown in elevation

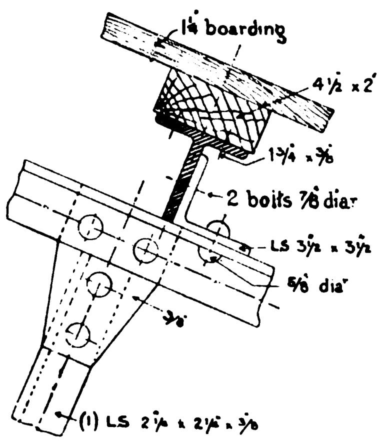


FIG. 304.
Scale 1½ inch = 1 foot.

in Fig. 251, and the side elevation of the gutter is shown in Fig. 297, which shows the position of the flanged joint of the gutter

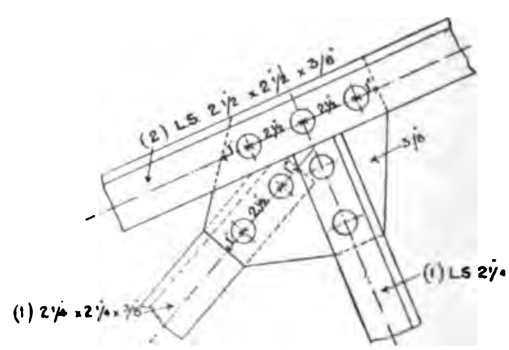


FIG. 305.
Scale 1½ inch = 1 foot.

with respect to the principal, the connection being made a few inches away from the truss, which simplifies detail and offers facilities in erection.

The detail of flanged connection to the wall gutter is shown in Fig. 259.

The roof principal shown in Fig. 216, in connection with the assemblage of columns and traveller girders, etc., there shown, is

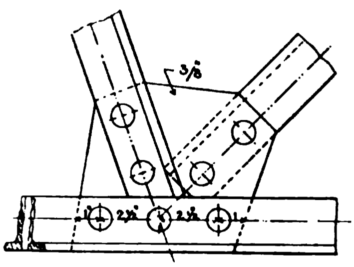


FIG. 306.
Scale 1¼ inch = 1 foot.

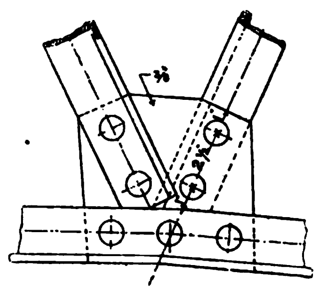


FIG. 307.
Scale 1¼ inch = 1 foot.

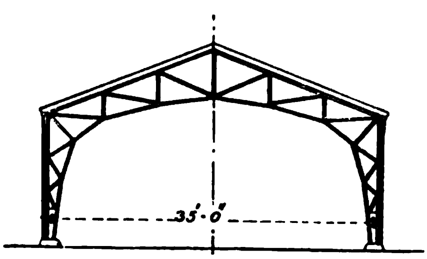


FIG. 308.
Scale 1 inch = 20 feet.

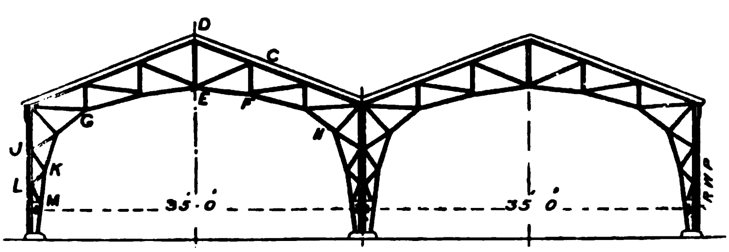


FIG. 309.
Scale 1 inch = 20 feet.

composed entirely of angle steels, the upper member consisting of two angles, the lower or tension member also of two angles and the bracings of single angles. The connections are all riveted.

The details of this truss are shown in Figs. 298 to 307 inclusive.

Fig. 298 shows the connection of a pair of principals, supported on the upper flange of a lattice girder (shown in detail in Figs. 81 to 101 inclusive), with their shoes and the valley gutter resting

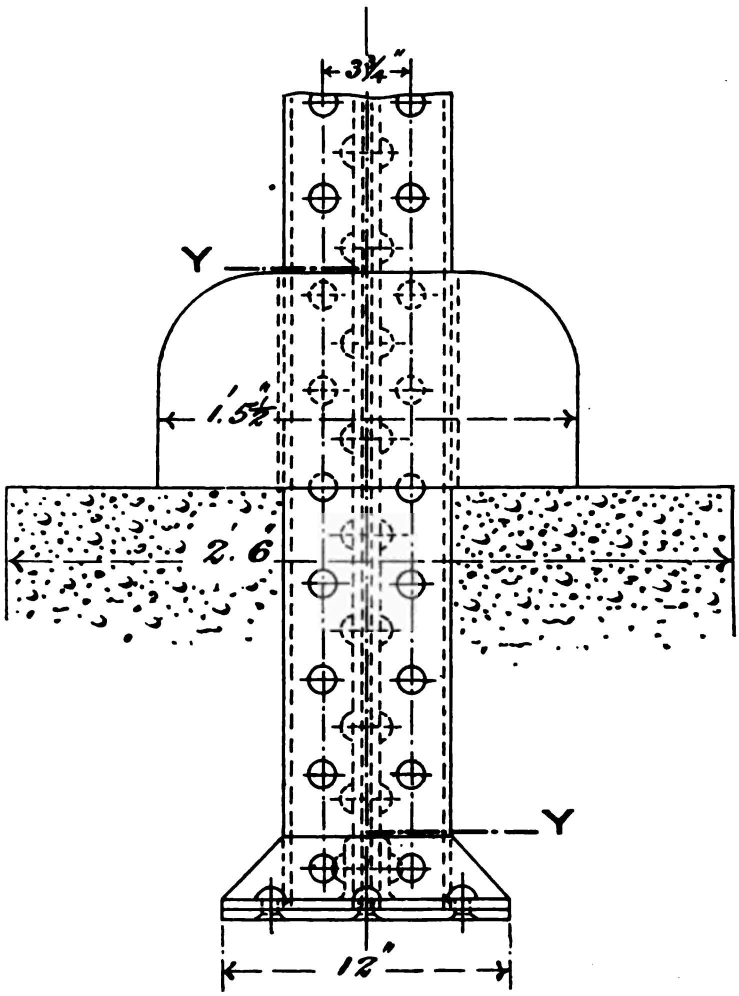


FIG. 810.
Scale 1¼ inch = 1 foot.

upon the backs of the principals. The shoe at eaves gutter is shown in Fig. 299, in plan in Fig. 300, and in sectional elevation in Fig. 301, the general arrangement being indicated in Fig. 302.

The detail at apex of principal, showing foot of central cast-iron standard, connection of ridge purlin, etc., is given in Fig. 303.

The riveted connection of the braces with the upper and lower members of the principal is shown in detail in Figs. 304, 305, 306,

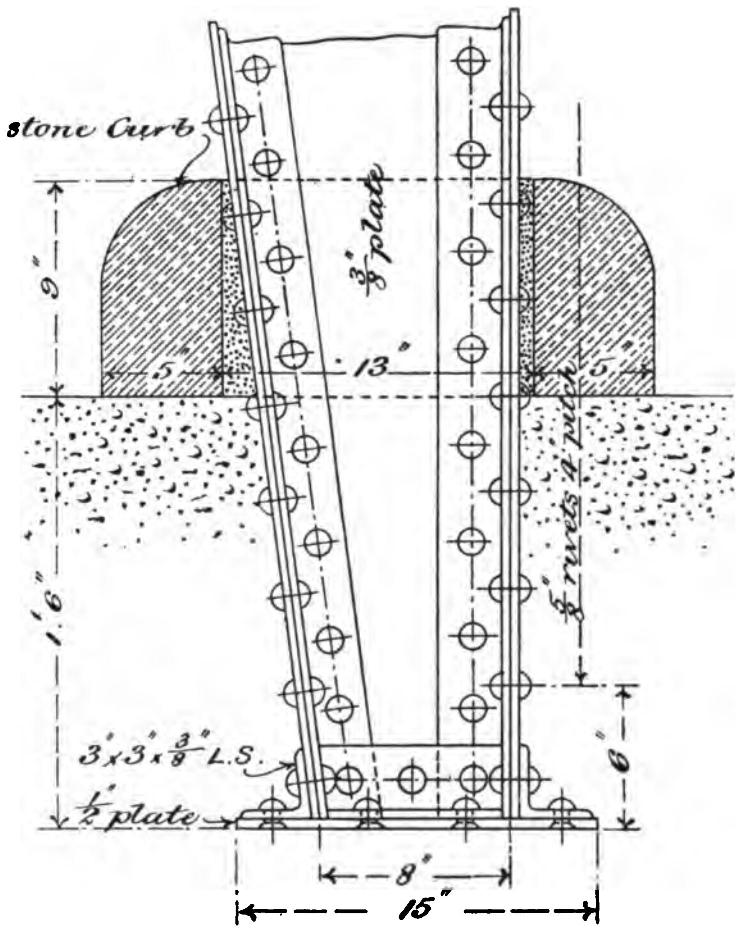


FIG. 311.
Scale 1¼ inch = 1 foot.

and 307, Fig. 304 showing the detail of connection of purlin to main rafter or upper member of principal. The purlins, which are of tee steels, are capped with a timber sill 4½" × 2", to which the roof boarding is nailed, and it may here be remarked that where

boarding is to be attached to ironwork, two alternatives appear to present themselves as a general rule—either the ironwork is

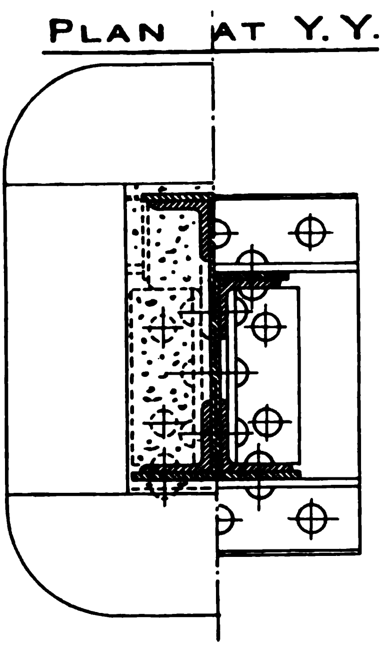


Fig. 312.
Scale 1½ inch = 1 foot.

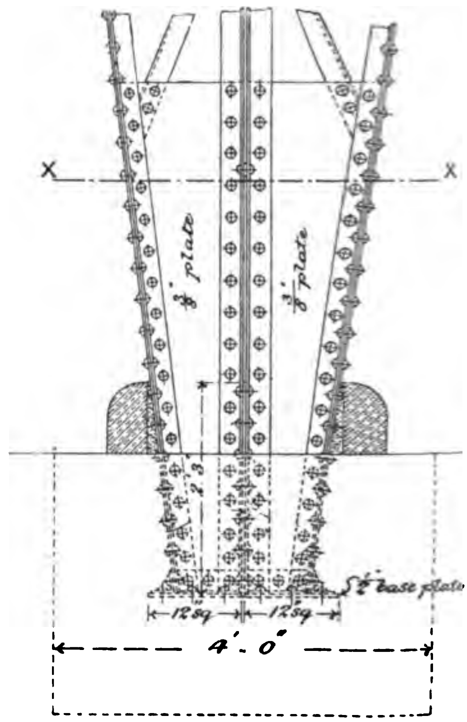


Fig. 313.
Scale ½ inch = 1 foot.

punched or drilled for screws to be attached directly to the boarding, in which case the pitch of the holes must be such as to

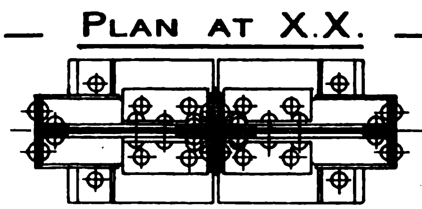


Fig. 314.
Scale ¾ inch = 1 foot.

agree with the width of the boards. This latter implies that the gauge of both holes and boards must be rigidly maintained, as, if

one gains on the other, a screw will fall on a joint of the boards sooner or later, which leads to bad work. The maintenance of such accuracy becomes difficult in practice, and the other alternative method appears the better, being the construction shown in the figure. The timber sill is attached to the top table of the purlin by screws at a pitch which need not have greater accuracy than such work implies, while the boarding is nailed or screwed

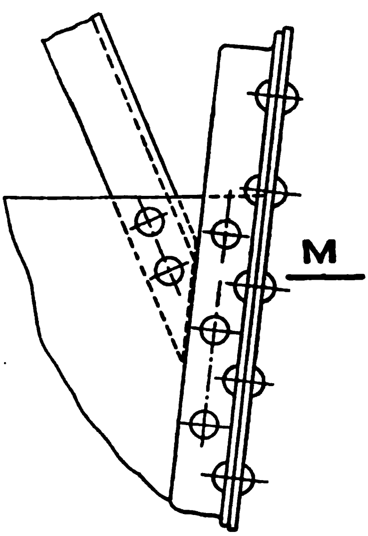


FIG. 315.
Scale 1¼ inch = 1 foot.

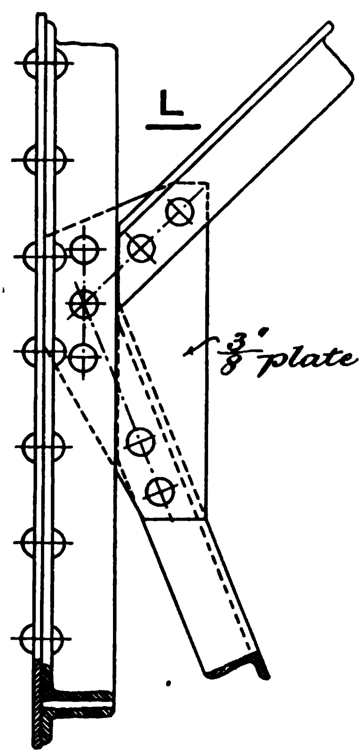


FIG. 316.
Scale 1¼ inch = 1 foot.

down, timber to timber, no particular maintenance of gauge of boarding widths being necessary.

Details of lantern and ventilator standards, with their attachments to rafters and purlins, and the junction of roof covering with ventilator details, are shown in Figs. 262, 264, and 265, 266.

The covering of this roof consists of slates on felt and boarding.

That class of structure which is represented by a series of roof principals supported on rows of vertical columns includes a multiplicity of examples applied to very diversified purposes, whether the structure be closed in or left open-sided and open-ended.

In this type of structure vertical loading is usually amply

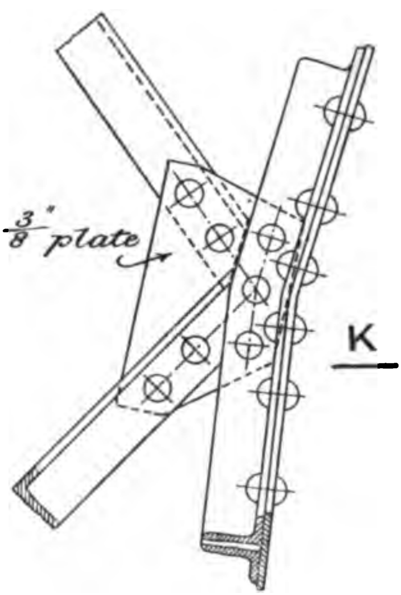


FIG. 317.
Scale 1¼ inch = 1 foot.

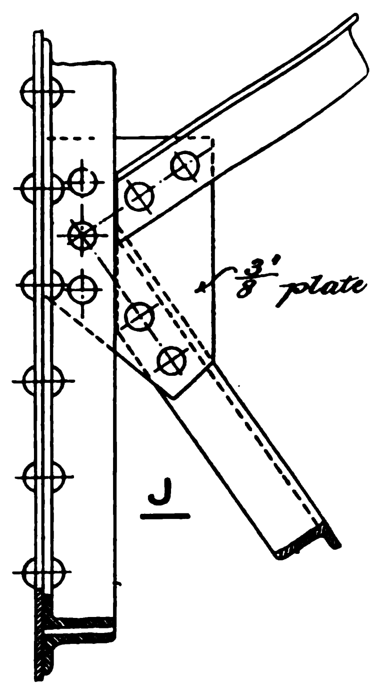


FIG. 318.
Scale 1¼ inch = 1 foot.

provided for, while the stresses due to horizontal wind pressure, or the horizontal components of that pressure, are resisted by the rows of columns, considered as vertical cantilevers, fixed at their bases.

This fixing of the base, including within that term the weight of the foundation to which the column is attached, together with the resistance of the attachments, holding-down bolts, or the like, frequently determines the ultimate resistance of the whole

structure to overturning, and instances are not wanting where failure has occurred, under exceptional wind pressures, by reason of the

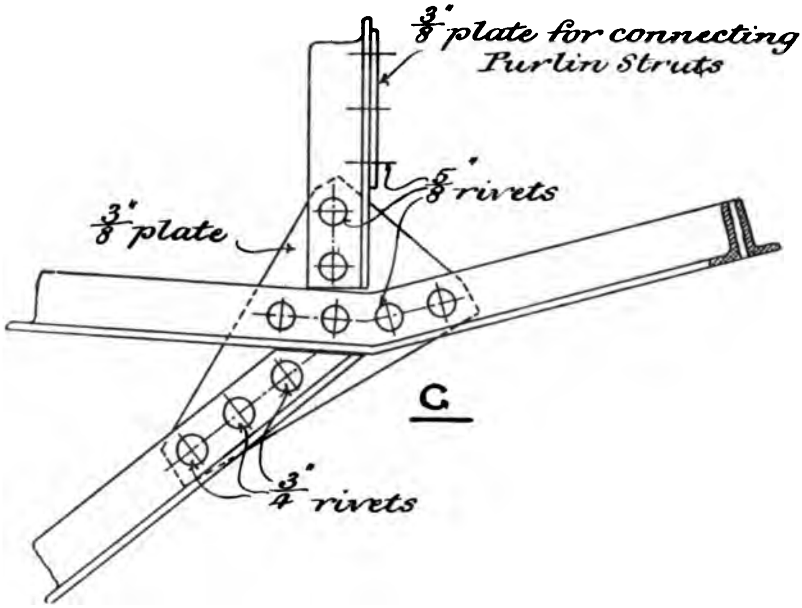


FIG. 319 (Scale 1½ inch = 1 foot).

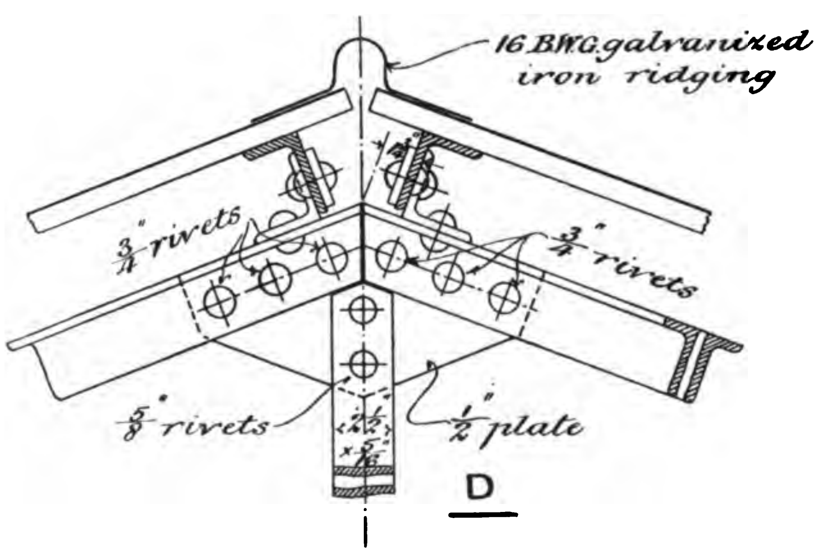


FIG. 320 (Scale 1½ inch = 1 foot).

overturning or pulling out of the ground of the foundation blocks to which the vertical standards or columns were attached.

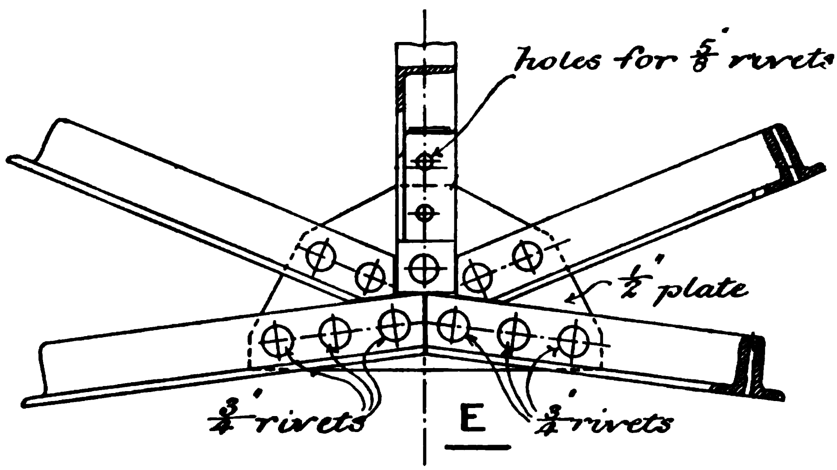


FIG. 321.
Scale 1½ inch = 1 foot.

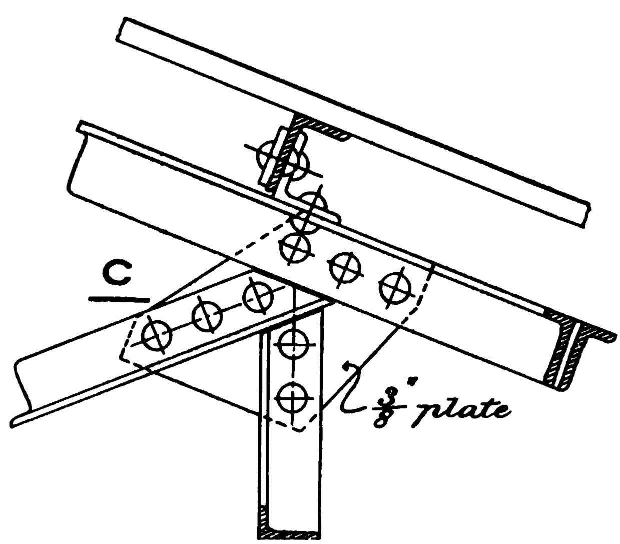


FIG. 322.
Scale 1½ inch = 1 foot.

Considerations of this kind may occasionally lead to the employment in certain cases and in exposed situations of the types

of roof principals represented in Fig. 308, where the roof principal is combined with the supporting columns in such a manner that

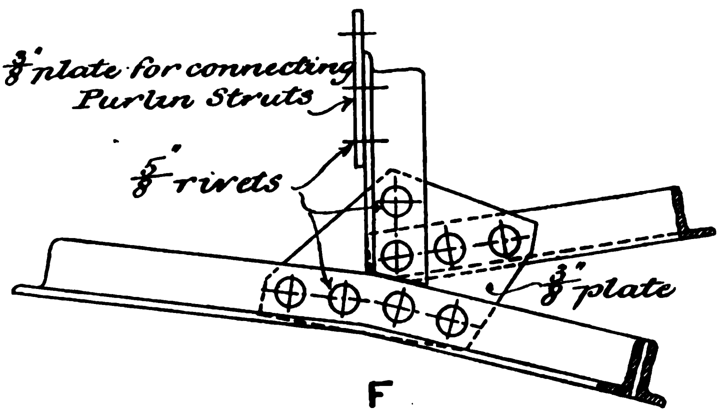


Fig. 323.
Scale 1½ inch = 1 foot.

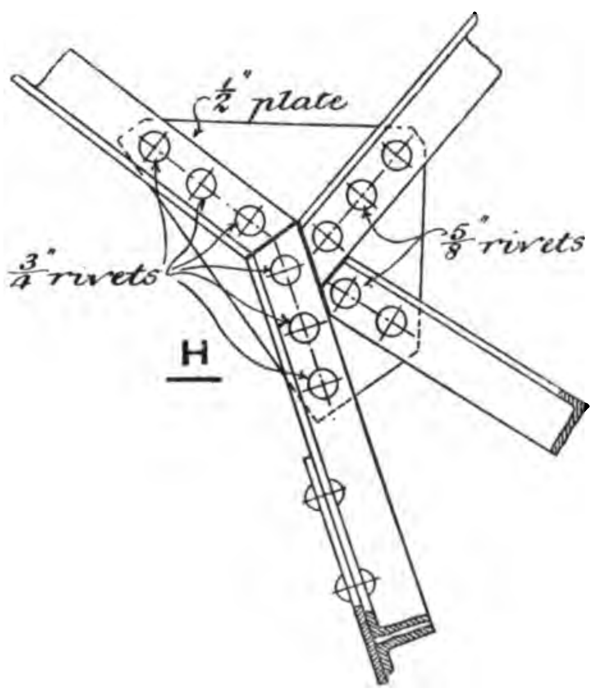


Fig. 324.
Scale 1½ inch = 1 foot.

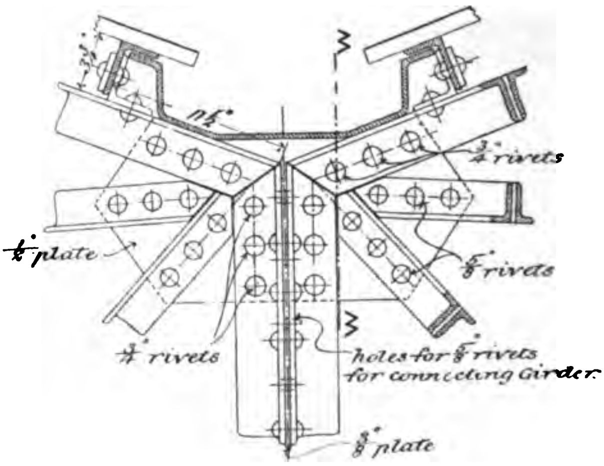


Fig. 325. (Scale 1 inch = 1 foot).

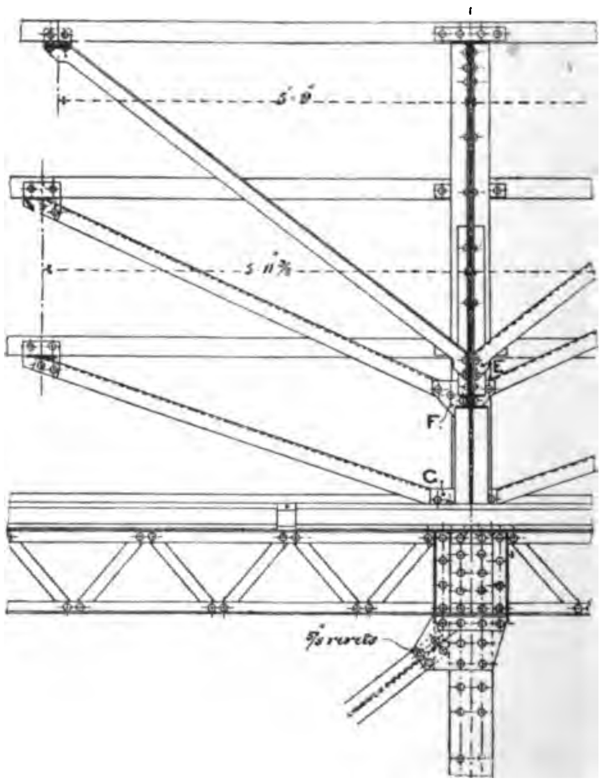


Fig. 326 (Scale $\frac{3}{8}$ inch = 1 foot).

the entire structure becomes virtually a rigid whole, capable of resisting satisfactorily the maximum horizontal wind pressures to which it can be exposed.

This type has been successfully employed in numerous instances, ranging up to considerable spans. While not so well adapted to the reception of traveller girders or other lifting

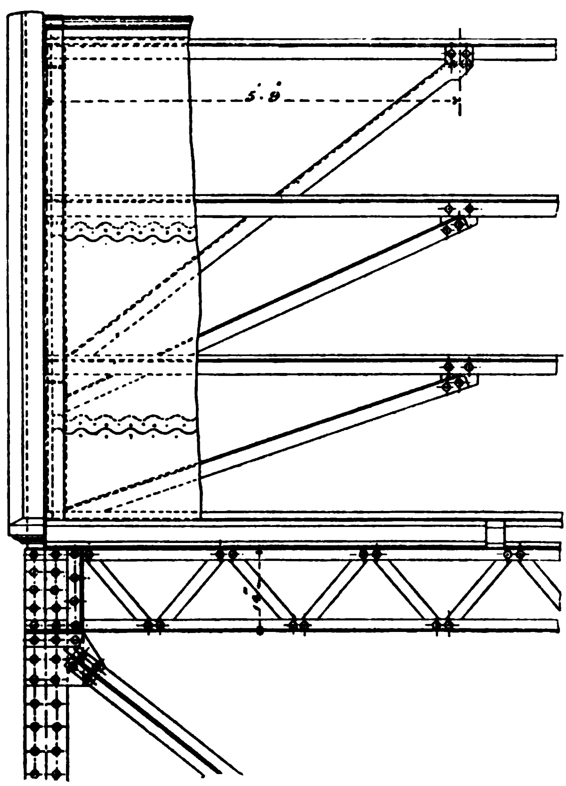


FIG. 327.
Scale ¾ inch = 1 foot.

appliances as other types, it is very suitable for such structures as drill halls, concert halls, markets, and the like, the internal appearance of the curved ribs being satisfactory, if left exposed, or if ceiled the curvature of the lower members can be adjusted to the desired amount.

The calculation of stresses due to combination of horizontal

and vertical loads can be easily made by the usual methods of graphic analysis or by the method of sections, and it will be usually found that the weight required in foundation blocks is but slight, and the cost of foundations, in ordinary soils, will thereby be decreased.

The example here given is only of very moderate span, 35 feet, but the details will be treated somewhat fully, as affording guidance

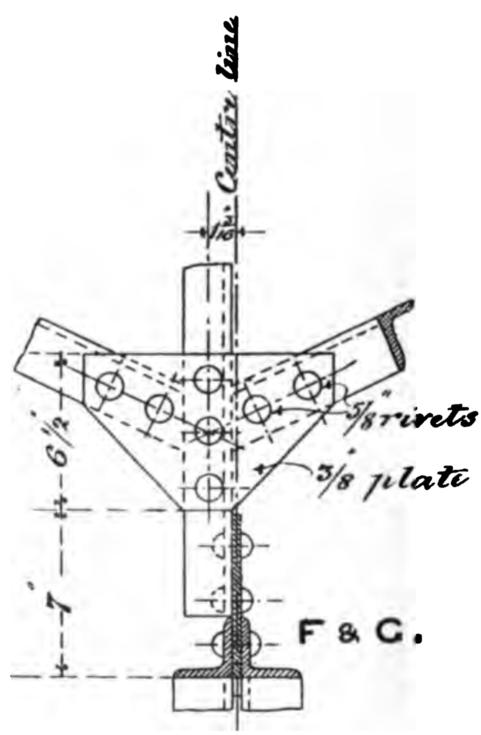


FIG. 328.
Scale 1¼ inch = 1 foot.

to the student in the design of a useful class of structure. As the purpose to which the building was applied did not require any enclosing walls, the sides and ends were left open, and as no roof lights or ventilators were consequently required, the roof covering details were of a very simple nature.

In Fig. 309, a pair of such structures is shown set back to back, forming a shed 70 feet wide in two spans.

The method of securing the bases of the standards in their foundation blocks of concrete is shown in Figs. 310 and 311, in front and side elevation. To protect the standards from the blows

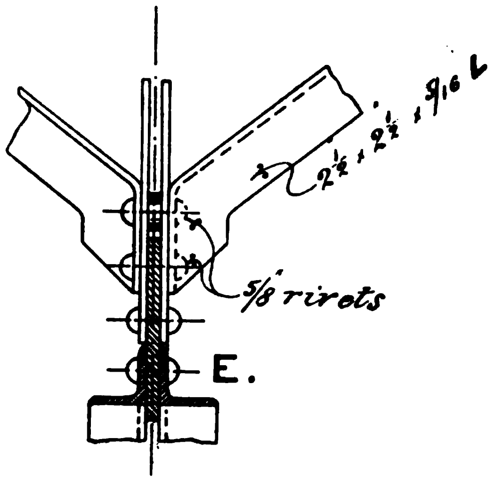


FIG. 329.
Scale 1½ inch = 1 foot.

of wheels of passing traffic, to which they are exposed, limestone curbs are placed, encircling the standards as shown in Figs. 310

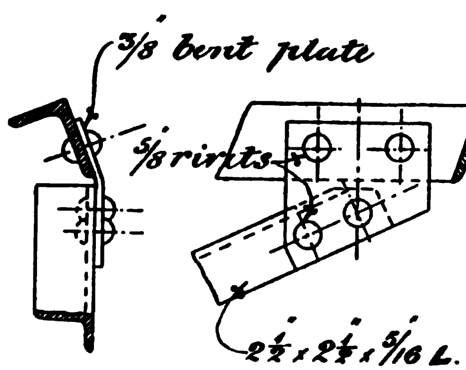


FIG. 330.
Scale 1½ inch = 1 foot.

and 311, and in half plan in Fig. 312. The base of the standards is not provided with holding-down bolts in this case, as it is not

capable of drawing out of the concrete block in which it is embedded, by reason of its shape. A sufficient thickness of

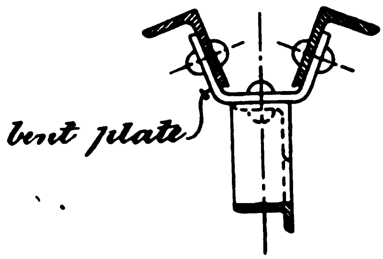


Fig. 331.
Scale 1½ inch = 1 foot.

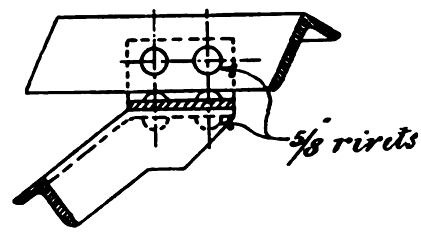


Fig. 332.
Scale 1½ inch = 1 foot.

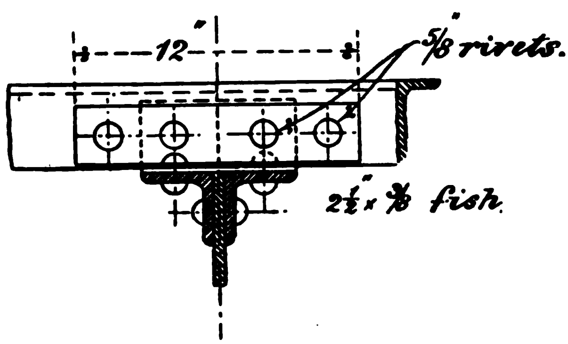


Fig. 333.
Scale 1½ inch = 1 foot.

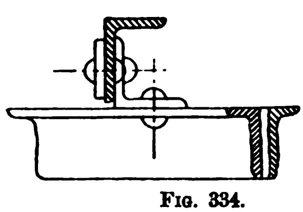


Fig. 334.
Scale 1½ inch = 1 foot.

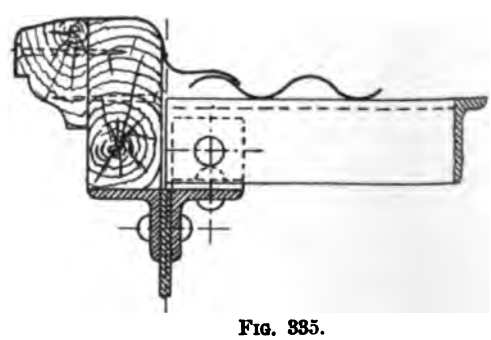


Fig. 335.
Scale 1½ inch = 1 foot.

concrete is left under the base to provide for vertical loading, to suit the conditions of soil on which the building is constructed.

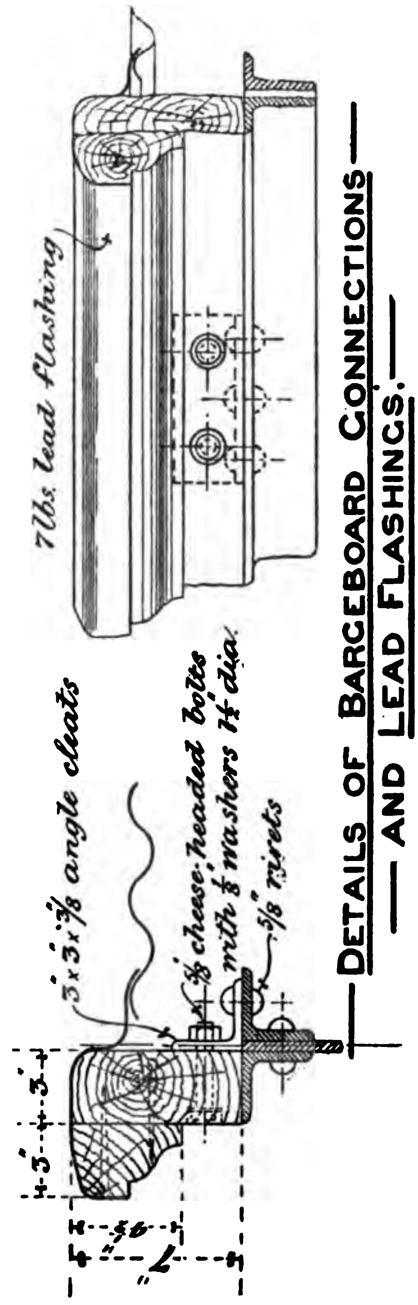


FIG. 336.
Scale 1½ inch = 1 foot.

A similar foundation with limestone curb is shown, for the centre column of the double shed shown in Fig. 309, in Fig. 313. A sectional plan on line XX is shown in Fig. 314.

The upper and lower members of the roof truss are each formed of a pair of angle steels, spaced sufficiently far apart to admit the sketch plates which form the bracing connections. The objections to this form of construction from the point of view of the paint brush are pointed out in Chapter VII., but the

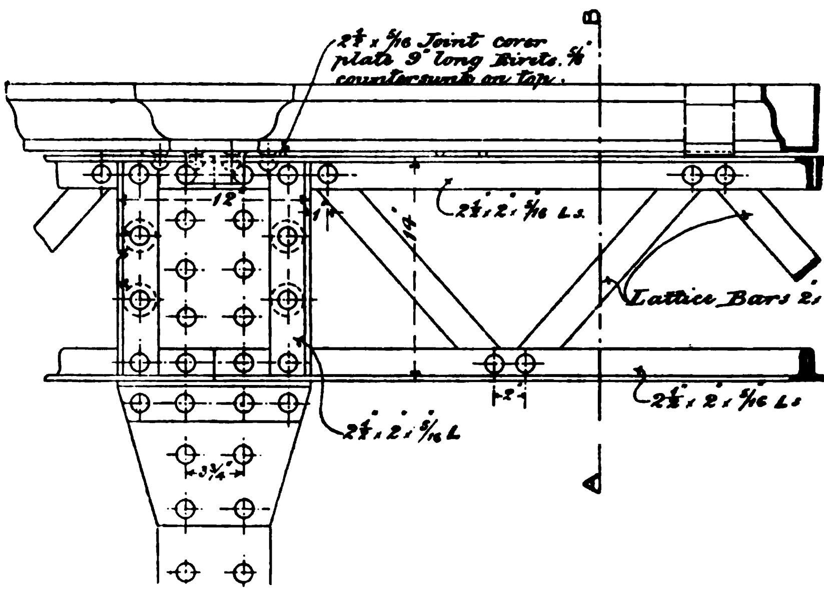


FIG. 337.
Scale 1 inch = 1 foot.

convenience and fitness of the arrangement in other respects is shown by the details of connections which follow.

Figs. 315 to 324 inclusive are details of the connections of the various points of junctions lettered in the figures, the same lettering being employed in Fig. 309.

Fig. 325 shows the detail of connections at the valley in the case of the pair of trusses placed back to back in Fig. 309.

It will be observed that a system of vertical members is introduced in the secondary bracing which affords facilities for the attachments of the struts to purlins. The purlins throughout the roof are of simple angle steel, but strutted in the manner

shown in Figs. 326 and 327, the principals being 17 feet 3 inches apart centres.

Figs. 328 to 332 show the details of connections of the struts to the vertical members of the truss and to the angles forming the purlin bars, while the details at joints of purlin bars and their

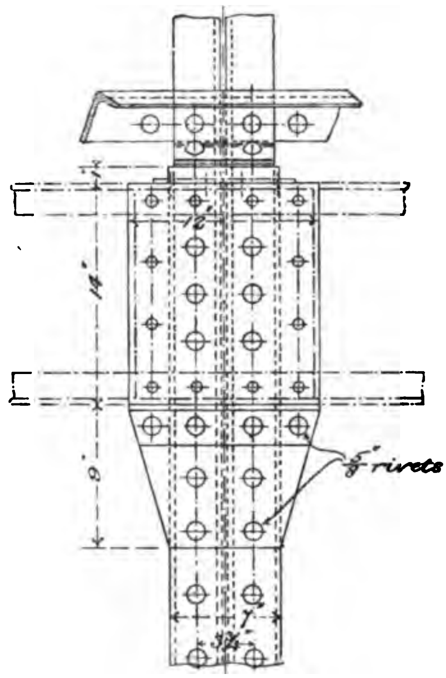


Fig. 338.
Scale 1 inch = 1 foot.

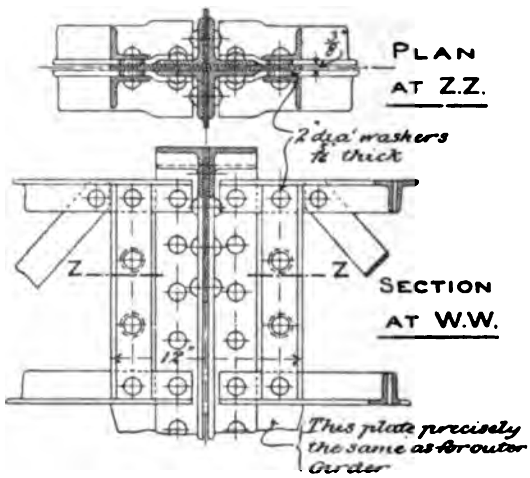
connection to the upper member of the truss are shown in Figs. 333, 334.

Figs. 335, 336 show the connection of end of purlin bar to end principal, and also the details of the barge-board arrangement over end gable, with connection between the corrugated iron roof covering and lead flashing over barge-board.

The caps or heads of the standards at the roof eaves are all connected longitudinally by lattice girders 14 inches deep, carrying the eaves gutter as shown in Fig. 337, while the detail of the head of the standard prepared to receive the lattice girder is shown in Fig. 338.

Similar details for the central lattice girder at the valley of the double shed shown in Fig. 309 are shown in Figs. 339 and 340, Fig. 340 being a section on the line WW shown in Fig. 325. Further details of the fascia and central girders are given in Figs. 341, 342.

A general cross-section of lattice fascia girder with eaves gutter resting upon it, and lowermost purlin bar, is given in Fig.



FIGS. 339, 340.
Scale 1 inch = 1 foot.

343, further details of eaves gutter and connections to lattice girder being shown in Figs. 252, 253.

The connections of eaves gutter to rain-water down-spout, and the mode of connection of the latter to framework of truss, are shown in Figs. 254 and 255.

Details of expansion joints, stopped ends, etc., of gutters are given in Figs. 256 to 260.

The detail of joint in valley gutter for double shed is shown in Fig. 257, while the method of attachment of valley gutter to the central lattice girder which carries it is shown in Fig. 258.

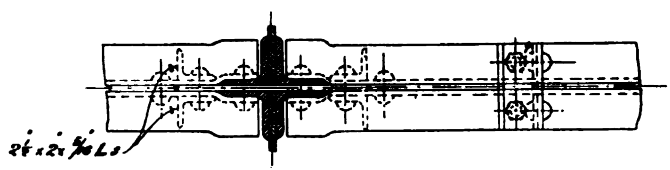
To ensure longitudinal stability in the entire length of shedding, certain bays were furnished with diagonal bracing (cruciform in arrangement) extending from the foot of one standard to the cap of the next. The attachment of bracing to the foot of a standard is shown in Fig. 344, and that to a cap of a standard in Fig. 326.

The covering of the roof was of galvanized corrugated sheet iron, attached to the purlin bars with hook bolts in the usual way.

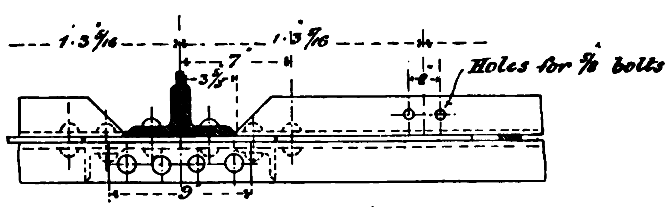
The type above described is of course equally applicable to sheds with closed sides and ends, while provision can be made if required for skylights, ventilators, or roof glazing by the necessary modifications in detail of purlins, and attachments of skylight standards, etc., to the upper member of the truss.

The Testing of Roof Principals, Girders, or other Structural Work in the Contractor's Yard.—It is not unfrequently stipulated in a specification for roof work that one or more bays of roofing

Fig. 341.
Scale 1 inch = 1 foot.



DETAIL OF CENTRAL GIRDERS FOR DOUBLE SHEDS.



PLAN OF FASCIA GIRDER.

Fig. 342.
Scale 1 inch = 1 foot.

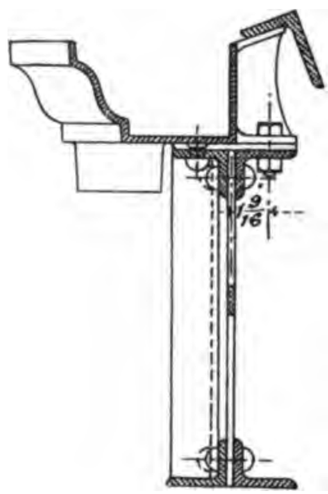
erected complete, as far as the ironwork is concerned, but without the roof covering, shall be tested in the contractor's yard prior to taking down and erection on the site, with such dead loads as shall represent those conditions of wind pressure or weight of snow which the roof principal has been designed to meet.

The loading is frequently adjusted so as to represent either an equal or unequal distribution of wind pressure, and is usually made up by a sufficient number of such sections of steel or iron bars as may be handily available for the purpose, laid across from principal to principal; or, it may be, in the case of lofty roof principals of large span, by pig-iron suspended by straps from

those points of the main rafter where the load arising from the purlins would occur in practice.

Where, as is usually the case, the actual roof covering is not present, then of course the weight per foot super of the covering must be allowed for, and added to the load representing wind pressure or snow.

It is customary, in the course of such a testing of the strength and stiffness of a roof principal, to ascertain the actual amount of deformation or deflection which the framed structure undergoes,



—SECTION A-B.—

FIG. 848.
Scale 1¼ inch = 1 foot.

and it may not unfrequently be the duty of the designing draughtsman to supervise the operation of testing, and to adopt such methods of practical measurement as will show such deformation or deflection. Such opportunities of witnessing the behaviour of any framed structure under conditions of actual loading should always be welcomed and made the most of by the student, who will find in them at all times instruction and interest, while possibly some disturbing thoughts on the accuracy of the results of his stress diagram may arise, if, for example, he

finds occasionally certain tension members manifesting a distinct inclination to behave as struts.

The conditions of practical testing of a roof principal or girder in the contractor's yard will, as a rule, be found to include, first, a more or less loosely adjusted platform of balks or sleepers upon which the steel work is erected, and forming the supports or abutments; and, secondly, a foundation stratum of the loose ashes so frequently forming the surface of an erecting yard, and which is to a large extent compressible and yielding.

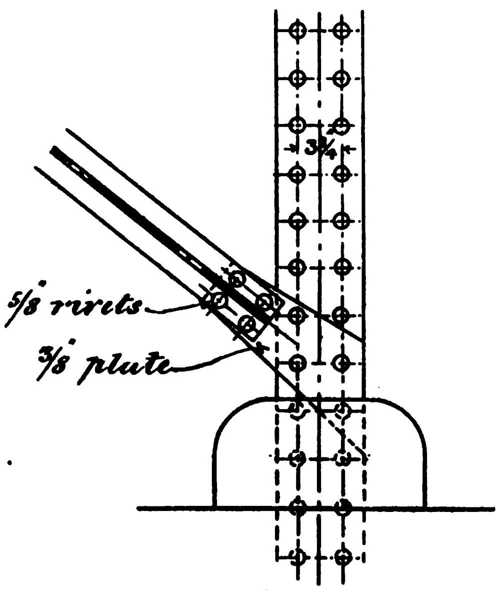


FIG. 344.
Scale ⅜ inch = 1 foot.

These two conditions taken together imply an amount of settlement of the supports which may possibly exceed in value the amount of deflection or deformation proper to the structure itself, and which is the result of the bending moments due to the loading. It follows therefore that vertical settlement of the temporary supports due to compression of loosely packed timber or soft soil, or both together, must not be mixed up with the measurements of the actual deflection of the structure itself due to span and loading, otherwise the results will be of little value.

There are various methods by which this little difficulty may be overcome, and the actual deformation of the structure ascertained. One of those very commonly adopted is to stretch a string or wire from end to end of the principal or girder, to serve as a datum from which careful measurements may be taken to the point or points at which the deformation of the structure is to be ascertained. If pegs (not liable to disturbance) are firmly driven into the ground on the line of the stretched string or wire, and the line of string marked thereon before and after loading, the total settlement can be ascertained.

Long strings or wires are, however, apt to be troublesome in wet and windy weather; and in certain classes of structures, such, for example, as the bow-string or sickle type of roof principal shown in Fig. 345, a better result may be obtained by the use of the ordinary spirit-level and staff, readings being taken at all convenient points, including, of course, the levels of the abutments or supports, before and after loading. A bench mark not liable to disturbance should be selected for reference and check. Where it is desired to ascertain the deflections of the upper member of the roof truss at points inaccessible to the staff, suspended wooden rods, slung from the points referred to, may be used, the intersection of the cross wires of the level being marked upon the rods before loading.

The whole series of levels being reduced to a common datum, and the settlement of the entire structure (as apart from the deflections due to bending moments) being ascertained, the actual deformations can be calculated, as though the structure had been placed upon an unyielding foundation.

Roof principals of considerable span show a tendency to stretch, due to elongation of the main tie, and this small increase of span should be watched and measured.

If the centre lines of main tie and principal rafter do not coincide at the shoe, but at some point outside it, certain other indications of strain will possibly become apparent, of which the student should take note.

The amount of actual load to be piled upon the rafters of a roof principal, of any considerable span, to represent an assumed wind pressure of, say, 40 to 50 pounds per square foot of surface, is very considerable, and in cases where only two principals, or one bay of roofing is erected, possibly without bracing, care should always be taken to avoid any risk of failure by lateral flexure of the principals, a type of collapse to which principals are

particularly exposed, and which has led to sometimes disastrous failures in course of erection.

This can be obviated by sufficient lateral strutting, but such support should be so arranged as not to carry any component of vertical test loading to the relief of the principal, but only to act as a safeguard against lateral flexure.

In connection with the subject of the behaviour of principals under test load, some consideration may here be given to the disputed question as to how far keys and cotters, or screwed connections, are advantageous or otherwise to the braced structure.

Custom has, to a large extent, sanctioned the employment of keys, cotters, or coupling screws in the main ties of roof principals of all spans, apparently with the idea that such adjustments are desirable or necessary in the final regulation of the exact span between end connections or shoes.

It is perhaps doubtful whether the ordinary methods of careful setting out and good templet work do not, in the yard, secure an amount of practical accuracy sufficient for all such requirements, when the actual dimensions of the work to which the roof is to be attached have been, as they should be, accurately ascertained and rigidly adhered to in construction.

It may again be argued that the use of keys, cotters, or coupling screws may lead to considerable modifications in the stresses in the members thus connected, when the adjustable connection has been used in such a way as to over- or under- strain the members affected.

A case of this kind came under the writer's notice during the testing of a roof principal of the type shown in Fig. 345. Keyed and cottered connections were used at the points A, B, C, D, E, F, G, and H.

Upon the application of the test load, which was suspended from the points J, K, L, M, N, O, P, the results of observation showed evident traces of undue stress in the members PB and GJ, a considerable proportion of the tensile stress which should by theory have passed through the members AB and GH having been diverted through PB and GJ, with the result of deformation in the top boom at P and J, these bars having to be reset to their original curve, while some elongation of rivet holes in connections at P and J showed the direction and intensity of stress.

It was considered these results were probably due to an in-u dicious use of the hammer by the workmen employed in erecting

the principal, who found the keys and cotters, to their thinking, too loose.

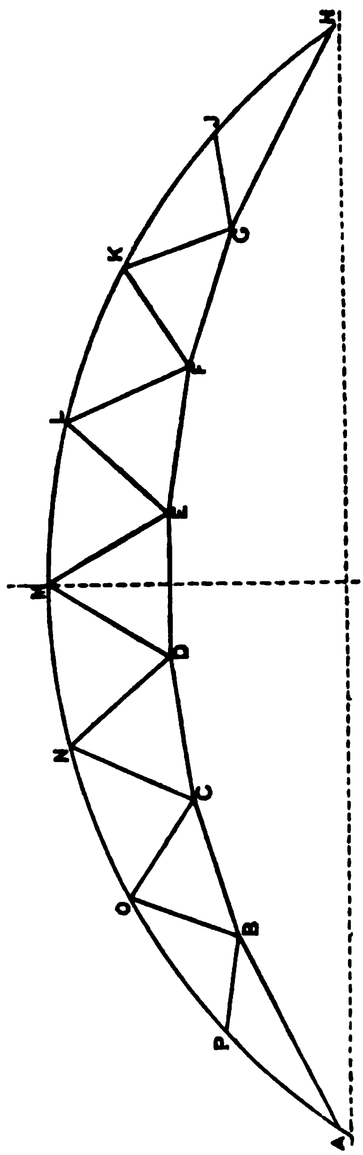


Fig. 345.

Incidentally, the result appeared also to point to the desirability, in designing roof trusses of this type, of avoiding such an inclination of the end braces in relation to the main tie as would, by accidental misfit, lead to serious modification of the calculated stresses.

In concluding this chapter, some brief reference may be made to the subject of setting-out roof principals of considerable span, and for illustration the type of truss shown in Fig. 345, and already referred to, may be taken.

This truss was used to cover an area unequally divided into two spans, portion of which was rectangular, and the remainder on a curve.

The actual distance between the seatings of the principals on the bed-stone course of the main walls of the building was ascertained trigonometrically by the use of a theodolite having a metal wall-tripod, and from a measured base laid out on the top of the wall itself. By these means the exact value of all the varying spans was obtained from the masonry work as built.

The upper member of the truss was a segment of a circle, and the centres of eye-bar pins in the tension member, or main tie, were set out also on a circular curve of larger radius.

The dividing up of the upper and lower chords having been determined, the exact lengths of the braces were calculated trigonometrically.

The entire truss was then carefully set out on a wood platform or scribing floor, the centres of all geometrical intersections being marked by gun-metal wood screws, having a small conical hole drilled in the nick in the centre of the screw head. These screws could be set with great accuracy by slight taps of the hammer while being screwed in, and were not subject to after-disturbance. The points of the trammel bars being adjusted to the conical holes in screw heads, the exact dimensions, say centre to centre of eye-bar heads, were accurately transferred from the setting-out floor to the drilling machine, and the important fiducial points being in metal, wear and tear were reduced to a minimum, and mechanical accuracy was maintained to the last.

The mode in which the erected work finally came together was proof of the soundness of the methods employed.

Such a setting-out floor as above described should be under cover, as if exposed to the weather the accuracy of the work in large structures may suffer, and a straight line drawn obliquely across a series of parallel boards with line and pencil one week, may the next week exhibit a lamentable falling off from its previous accuracy.

CHAPTER VI.

THE USE OF MILD STEEL AND IRON IN MARINE ENGINEERING.

General remarks—Design of iron piers or jetties—Classification of jetties—Those which are backed up by solid structure in the rear—Those which are isolated—Types of design of the second class—Accessories of jetties—Exposure and "fetch"—Height of deck—Details of pilework as influenced by conditions of soil or sea bottom—Modes of sinking piles—Difficulties as to finished lengths and levels of top of piles—Make-up lengths—Objections to make-up lengths—Sawing off of steel or iron sections—Composite structures of cast iron and timber—Example and description—Bollards—Crane roads—Floating booms—Description of cast-iron cylinder jetty with superstructure of steel girder-work—Cylinder spacing—Main box girders—Cross girders—Details of cylinders—Flange joints—Cap—Make-up lengths—Lowermost rings of cylinders—Cutting edge—Further adaptations of steelwork in marine engineering works—Tie-rods to wharf walls—Accessories to jetties or wharves—Bollards—Strength of bollards—Details to suit special cases—Anchorage—Examples—Fairleads and capstans—Position and arrangement—Foundations—Caissons—Varied applications of the term—Caissons for dock entrances—For foundations—For the commencement or completion of breakwater structures—Practical example of a caisson for breakwater construction—General description—Dimensions—General arrangement—Compartments—Bulkheads—Details of construction—Valves—Mooring rings—Method of erection and launching—Launching ways—Completion after launching—Ballasting—Weights—Conditions of stability—Composition of ballast—Subsequent operations—Towing into position—Sinking—Programme for concreting and successive conditions of stability—Titan cranes for block-setting—Caissons for closing dock entrances—General comparison between lock-gates and caissons—Types of caissons—Sliding—Floating—Comparison of types—Advantages and disadvantages—Functions as a bridge and as a dam—Watertightness—Design of keels and stems—Arrangement of sealing timber for floating caissons—For sliding caissons—Width of water seal—Details of timber keels and mode of connection, etc.—Consideration of pressures, reactions, and general stresses due to water pressure—Position of decks and air-chamber—Effects of corrosion and considerations of minimum scantlings—General description of internal arrangement of floating caissons—Bilge—Air-chamber—Stability and pendulum—Upper and end chambers—Scuttling tanks—Upper deck and top tanks—Dangers arising from excess of buoyancy—Critical conditions—Holding-down apparatus—Frictional resistance to uplifting—Coefficient of Friction—

Calculation for buoyancy and weight—Rivet heads—Percentage to be allowed —Weights of immersed timber—Subsidiary items—Table of the weights of recently constructed caissons—Ballast and stowage—Composition and densities of ballast—Burr concrete ballast—General description of sliding caissons —Arrangements of upper deck and camber deck—Sliding ways—Hauling mechanism—Additional resistances due to currents or differences in water level—Culvert area—Extra sluices—Position of hauling gear—Tilting moments —Sledge runners and rollers—Combination of the two types in recent caissons —Keels—Roller paths—Rollers and rams—Mode of operation during inward and outward journeys—Details of roller path and roller—Of hydraulic rams and mud-scrapers—Handrailing—Hauling chains—Description and manufacture—Tests—Tables of results of tests—Modulus of elasticity of hauling chains—Table of the weights of sliding caissons.

THE contents of this chapter are devoted to a brief reference to that branch of construction in mild steel or iron which may be entitled marine, as it deals with structures (other than ships or shipping) which are constructed either in the water, such as jetties or piers, or with reference to the requirements of navigation and commerce, in docks and dockwork generally, with their accessories, such as caissons, lock-gates, bollards, penstocks and the like, while an important branch of this subject is occasionally found in the application of steelwork to certain structures connected with the construction of breakwaters, an example of which will be illustrated and described later on.

The details of design of iron piers or jetties will vary greatly with the conditions which such structures are called upon to meet.

Thus, for wharves constructed for the loading or unloading connected with an extensive shipping industry, and alongside which ships of deep draught and heavy displacement have to lie, will have their details largely determined, amongst other ruling considerations, by the amount of resistance to be offered to the shock of heavy vessels coming alongside, in addition to the heavy strains brought to bear upon the structure through the bollards, capstans, or fairleads, by the hawsers used in bringing ships alongside, or getting them away.

In this respect piers or wharves may, for convenience, be classed under two heads—those in which the jetty or wharf is backed up by a solid structure, such as a wharf wall or rubble mound, and those in which no such supports are found, the structure having to resist the shocks or bumps of floating craft alongside without the aid of any such accessories, these stresses being in the former class transmitted to and absorbed by the solid structure in the rear.

It is obvious that in the second class much more consideration will be necessary as to the means by which such stresses are to be resisted, and the designer will usually find himself called upon to select either one of two types, that in which the supporting columns are numerous, spaced fairly close together, and connected by numerous braces, both transverse and longitudinal, converting the entire structure into what may be termed a basket-work of more or less elastic members, tending to absorb, by reason of their number and connections, the shocks or bumps of craft alongside, or the stresses caused by hawser connections.

The other type is that in which the vertical members are spaced further apart, and are of such dimensions and weight as may reasonably be expected to resist the shocks they may be called upon to endure, assisted by a heavily framed floor, and such bracing as it may be practicable to introduce. This type is of more rigid and unyielding character, and its individual members will probably derive less mutual assistance than the other, and in consequence the separate columns must, as before said, be of considerable weight and solidity. Such a type is commonly associated with supporting columns of cast-iron cylinders of considerable diameter, and which may at times attain the dimensions of bridge piers.

In either type it will, of course, be necessary to consider not only the amount of deck load to be carried, but the numerous accessories to be provided for on the deck of every commercial wharf or jetty, such as foundations for fixed cranes, the roadways for travelling cranes, the attachments of bollards or fairleads, while in jetties for the loading or unloading of coal special consideration will be necessary in the provision of coaling appliances, such as hoists, coal tips, coal spouts or hoppers, and the like. A railway system will be found frequently associated with such structures, and must be provided for in the design of the deck, including possibly framing for turntables, while the necessary requirements of electric, hydraulic, compressed air, or steam power will each demand separate consideration in connection with the conveyance of such power by pipes or cables, in troughs or otherwise, from a central station to the points at which the power is to be applied.

Commercial jetties or wharves, of which the general requirements are as above sketched, are usually found in waters more or

less sheltered, and the questions of sea-exposure and the "fetch" and dimensions of waves are of less importance.

But in the case of jetties or piers on exposed coasts, such as the familiar type of the promenade pier at our fashionable watering places, conditions other than those above described prevail, and it becomes of vital importance to consider the maximum height of wave likely to be experienced, and the minimum height above high water of spring tides at which the promenade deck should be laid to secure safety from the wrecking and lifting power of the crest of the wave, the security of the structure as a whole against wave action being derived principally from the small dimension of the vertical supports or columns, and the correspondingly small resistance offered by them to the advancing wave, a security which, however, may become grievously imperilled if the wave bears with it anything in the shape of wreckage, floating logs, or the like.

In such structures, the decking arrangements are of a simple character, the purpose of the pier being less commercial, the size and weight of the craft alongside being possibly not more than that of a loaded excursion steamer, the condition of stress arising from shocks or bumps are less severe (except so far as they may be affected by sea exposure), although it is customary to relieve the slender structure of such piers from this kind of stress by providing a totally independent landing jetty, so arranged as to take the shock of steamers coming alongside, to the relief of the more slender structure behind.

But whatever may be the type of jetty, wharf, or pier to be adopted, the details of the supporting piles or columns will always be largely influenced by the nature of the strata of the sea-bottom into which they are to be driven, screwed, or by other means brought down to their foundation level.

The subject of the measures to be adopted to force down the supporting piles or columns of a jetty, or the piers of bridgework, and the various precautions to be adopted in meeting all the contingencies arising from very variable qualities of strata, with widely differing resisting and bearing powers, and of varying degrees of watertightness, is of far too wide a scope than to be more than hinted at here.

The procedure may include ordinary pile-driving as applied to iron piles, the use of screw piles, either with or without the water jet, the combination of screws with a serrated cutting edge, the forcing down by dead weight loaded up on the top of the pile or

cylinder, combined with the removal of the soil inside either by means of a "miser" or in larger cylinders by a grab, the use of the "disc" piles with a water jet,—these and other methods, including the use in important cases of the compressed air system, will each and all take their share in the final results, while the application of any one of them will depend in all its details upon the variable nature of the local conditions which may have to be met, and in relation to which no general law can be laid down as to the best system to be adopted in any one particular case.

Thus, for example, the adoption of the screw-pile system will at once bring to the forefront the design of the screw-blade to be adopted, its diameter, scantlings, pitch, and methods of attachment to the body of the pile. For soft soils, where a considerable bearing area of blade is required, the diameter will be of as large dimensions as is consistent with safe screwing and handling, although in such cases it is very essential that the strength of the blade section is sufficient, considered as a cantilever, to prevent its breaking off from the body of the pile, or becoming sheared either by vertical stress or torsion.

On the other hand, in certain hard soils the blade diameter may have to be reduced to comparatively small dimensions in order that the labour of screwing may be kept within the bounds of what can be practically applied, and that the connections of the pile and screwed portion may not be sheared off, as sometimes has happened, by excessive torsional stresses.

In certain cases of exceptionally soft soils it has been found necessary to supplement the bearing area of the screw-blade, or the frictional resistance of the pile itself, by timber platforms so constructed as to distribute the pile load over a considerable area.

It may not unfrequently be necessary to determine, as the work proceeds, experimentally, the best proportion of pitch and diameter of blade to suit any given condition, and where this is likely to be the case, the provision of a separate casting for the blade, attached to the main body of the pile, but separable from it, obviously offers some advantages.

Cast steel is sometimes used for screw-pile blades. In this connection, as in many others, it will not be unwise to remember what does and what does not constitute cast steel.

The combination of the water-jet method, found so successful in sandy soils, associated either with the screw or the disc pile, will cause a variation in design in the arrangement, diameter, and

number of the jet holes to be provided for, while the attachment of the pump hose to the top of the pile will also receive attention.

As regards the sinking of cylinders of sufficient size for workmen inside to handle and remove the excavated material, nothing need be said here. The usual methods of applying the compressed air system, the use of the air lock, and the arrangements for the removal of spoil are well known and understood, while in certain cases where the soil entered is tenacious and impervious to water, the sinking of such cylinders by excavation and loading by dead weight can be carried out without the use of compressed air, though this system is often preferred, even in such cases, as affording a safeguard against "blows" caused by sudden and unexpected changes in the nature of the strata passed through.

Whatever may be the type of pile or cylinder adopted, whether it be a solid steel or wrought-iron column of a few inches diameter, or whether it be a cast-iron or riveted cylinder several feet in diameter, the difficulty of ensuring that the upper surfaces of a row of such piles or cylinders shall be in one true level plane after the operations of sinking and testing have been completed, will generally present itself for solution. The operation of pile or cylinder sinking is not in itself one which affords much room for fine adjustments, while the uncertainties which frequently attend the depth to which a pile must be sunk, either in order to reach a firm stratum or to afford such frictional resistances as will carry the load in soft soils, will often render the actual depth to be reached a doubtful quantity.

Where the superstructure of a jetty or wharf is of timber, a certain amount of cutting or packing to get over small differences of finished level may be permissible, but where a decking is of the more rigid type of riveted steelwork, a higher standard of accuracy in level is necessary, and this is frequently attained by the use of make-up lengths, the precise dimensions of these lengths being obtained from the work itself as soon as the operations of sinking, test loading, or concreting have been completed, and the pile or cylinder has reached its final depth and level. By this means the upper surfaces of piles and cylinders, with the seatings of the girderwork which they support, are maintained at their true levels.

An objection which may make itself felt, especially where the work is being carried out abroad, lies in the possibility of undue delay in the time taken in the transmission home of the necessary

dimensions, and in the manufacture and delivery of the special lengths of pile or cylinder. As regards certain classes of piles, this objection has been met, where the dimensions admit of it, by the mechanical sawing off on the spot of the redundant length of piles in iron or steel.

Thus, to take an example, solid mild steel or wrought-iron piles, say 6 inches in diameter, in long lengths, arranged for driving through more or less compact strata until a hard or rocky bearing stratum is reached, are sometimes provided with a pointed end formed in steel of somewhat higher temper than the body of the pile, though capable of being welded on to the softer material, while the provision for cutting off, after driving and test loading, may consist of a cutting tool arranged to travel circumferentially round the body of the pile, and driven by worm-gearing, impelled either by manual labour or other power.

Such a tool would, however, in the case of the so-called Phœnix column pile (shown in Fig. 174), be less advantageous than some form of circular saw, which would first attack the projecting flanges before arriving at the body of the metal.

In short, the cutting-off apparatus must be suited to the form of cross-section to be dealt with.

In the case of large cast-iron cylinders, however, the system of making-up lengths appears the only practicable one to adopt.

An example of a make-up length of this type is given in Figs. 358, 364, and also in Fig. 34.

Of the many types of jetty construction which might have been illustrated, space can only here be found for a few figures exemplifying a class of jetty which is composite in construction, that is to say, having an ironwork substructure and timber superstructure.

A few illustrations will also be given further on showing details of that type of jetty referred to on p. 339, where a rubble mound, or a wharf wall, or a combination of both, is faced with heavy cast-iron cylinders, spaced a considerable distance apart, and carrying a superstructure of riveted steel girders and decking.

The comparatively short life of timber in sea-water, and its exposure to the ravages of the teredo, frequently leads to the adoption of composite structures of this class, where the pile work is of cast or wrought iron up to and somewhat above low water, the remainder of the construction above this level being mainly of timber. The latter portion of the jetty, although still exposed to

the inevitable decay which takes place between "wind and water," is, at all events, accessible for repairs or renewal, the life of the iron piles below water-line being assumed to be considerably longer than that of the superstructure which they support.

In Fig. 346 is given a cross-section of a jetty belonging to the first of those classes discussed on p. 339, being supported in the rear by a rubble mound, which may be supposed to resist, or share in resisting, the shocks to which the structure may be exposed by floating craft alongside, and to which mound the jetty forms a

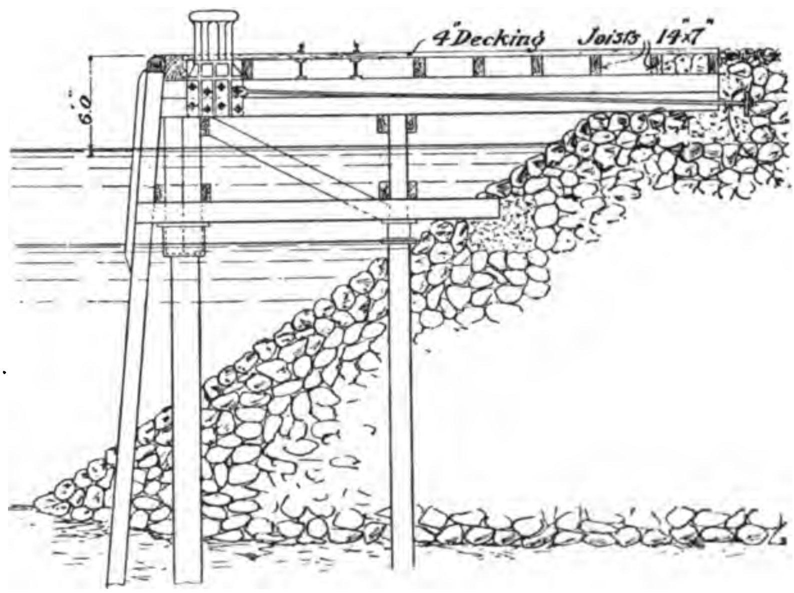


Fig. 346.
Scale 1 inch = 12 feet.

facing, having a sufficient depth of water alongside for the class of vessel for which it is intended.

The jetty consists of two rows of cast-iron piles, as shown in the figure, the outermost sloping fender pile being of timber. The distance apart of the rows of piles lengthwise of the jetty is about 10 feet, while transversely they are pitched as shown in the figure.

The outermost cast-iron pile is 1 foot 6 inches internal diameter of 1¾-inch metal. Two separate systems of joints of the pile are adopted, one for joints below the ground line, the other

for joints above the ground line and below the level of the caps and sockets to receive the timber work.

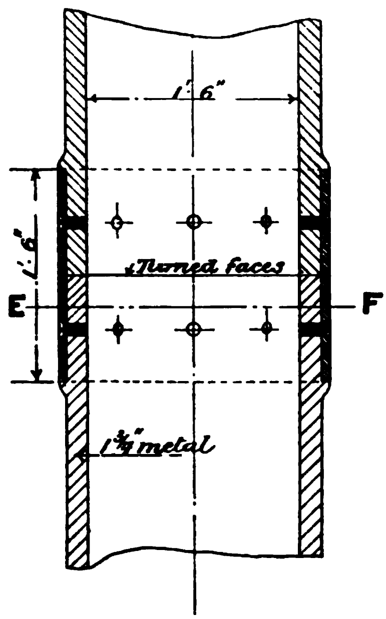


FIG. 347.
Scale ¾ inch = 1 foot.

The former joint is shown in Figs. 347 and 348, Fig. 348 being a section on line EF in Fig. 347. This method of jointing has been

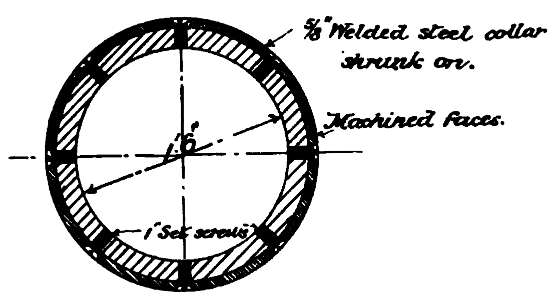


FIG. 348.
Scale ¾ inch = 1 foot.

found satisfactory for mud and sand, offering no resistance to the sinking of the pile. As shown in the figures, it consists of a

welded steel collar $\frac{5}{8}$ inch thick, machined on the internal surface, and shrunk on to the machined surfaces of the cast-iron pile, to which it is attached by 1-inch diameter set screws.

The details of the joint above ground line are shown in Figs. 349 and 350, this being an ordinary flanged joint with machined faces and drilled holes for bolts.

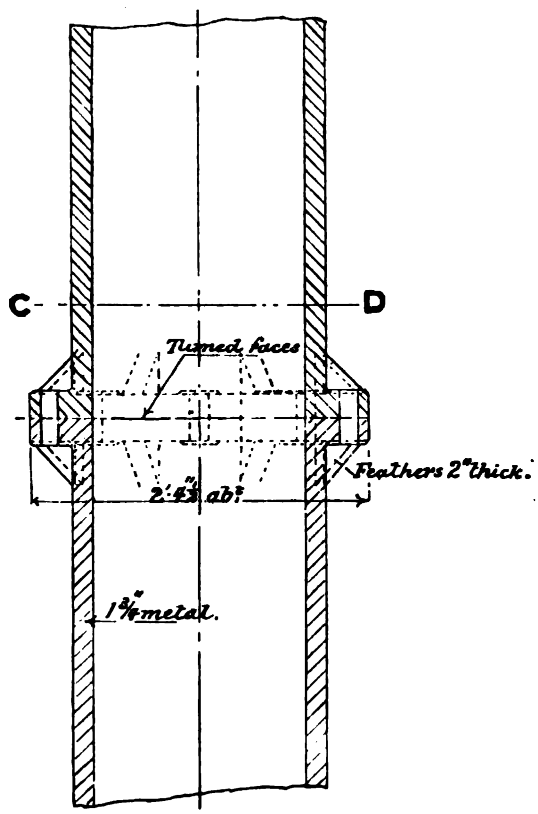


Fig. 349.
Scale $\frac{3}{8}$ inch = 1 foot.

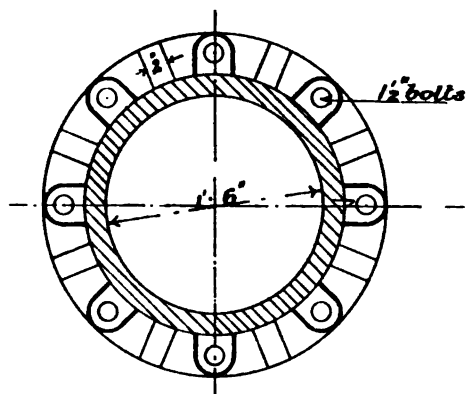
The detail at the top of the outer cast-iron pile is shown in Figs. 351 and 352. This consists of a cast-iron cap specially shaped to receive the feet of the double timber verticals of the superstructure, and socketed into the top of the circular pile below, as shown. The timber work is bolted to the caps by 1$\frac{1}{8}$-inch bolts, holes for which are provided in the castings, as shown.

The inner row of cast-iron piles is 12 inches internal diameter, of 1¼-inch metal, with joints similar in character to those of the outer row, and shown in Figs. 353, 354.

The caps of these piles are also arranged to receive the single timber verticals of the superstructure, as shown in Figs. 355, 356.

The deck or floor of the jetty is of 4-inch decking laid upon 14″ × 7″ joists, these latter resting upon the double timber girders shown in Fig. 346, which in their turn are carried by the timber verticals socketed into the caps of the cast-iron piles, as above described.

Horizontal and transverse walings, with struts, curbs, and



— SECTION.C.D. —

FIG. 350.
Scale ¾ inch = 1 foot.

capping pieces, complete the timber superstructure, as shown in Fig. 346.

Cast-iron bollards are attached to the double timber girders, and are anchored back to concrete blocks set in the summit of the rubble mound by means of long steel straps with keys and cast-iron washer plates.

Where a crane road occurs, steel girders are frequently inserted under the decking, to which the rails are connected by through hook bolts, as shown in Fig. 357, which represents a crane road on a timber jetty arranged for 20-ton cranes working at 25 feet radius, with a gauge of crane road of 11 feet centres of rails, as shown, the total weight of the crane without the load being about 80 tons.

The use of floating booms alongside jetties of openwork construction is not uncommon. It may in certain cases be necessary to prevent small floating craft, such as boats or barges, from being blown or pushed in under the girderwork of a superstructure which is not sufficiently high above high water to prevent the

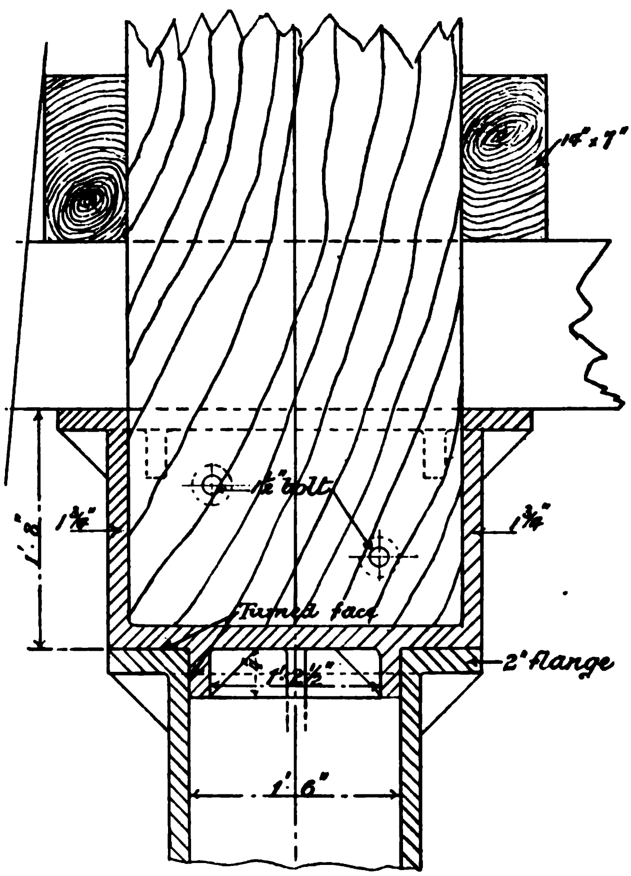


Fig. 851.
Scale ¾ inch = 1 foot.

risk of such craft being caught at the top of the tide, and by their displacement exerting an upward lifting force on the decking. Or, again, booms are used to mitigate the force of shock or rubbing as between the ship's side and the jetty face.

In the case of large cast-iron cylinders, circular rings of timber,

framed together, surrounding the cylinder and floating up and down with the tide, have been used, but are found to be severe upon the frames and skin-plating of the ship; on the other hand, if the boom is made straight, the nip then becomes visited upon the cylinder to the relief of the ship.

We may now consider the other type of jetty previously referred to, consisting of heavy cast-iron cylinders supporting a superstructure of main and cross riveted steel girders with decking,

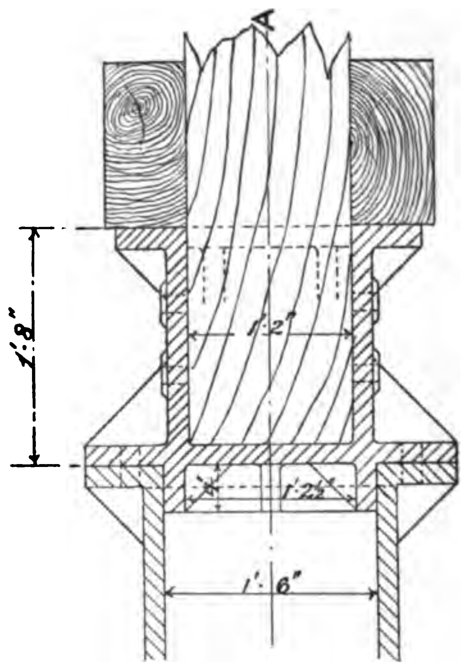


Fig. 352.
Scale ¾ inch = 1 foot.

the whole forming a facing in front of a rubble mound surmounted by a vertical wall in deep water, and adapted for the use of a heavy class of vessel of deep draught.

In the case before us the local conditions and the cost of sinking cylinders require that the latter shall be spaced as far apart as is practicable without unduly increasing the dimensions and weight of the main girders. This spacing is fixed at about 60 feet in the case shown in Fig. 358, which represents the upper portion of one cylinder and portions of the adjoining spans of main

girders. An enlarged view of the upper portion of the cylinder, showing the capping, the junction of the main girders, and the bollard attachment, is given in Fig. 359.

The main girder connecting the tops of the cylinders is of a heavy box-girder type, and is shown in section in Fig. 360. Cross

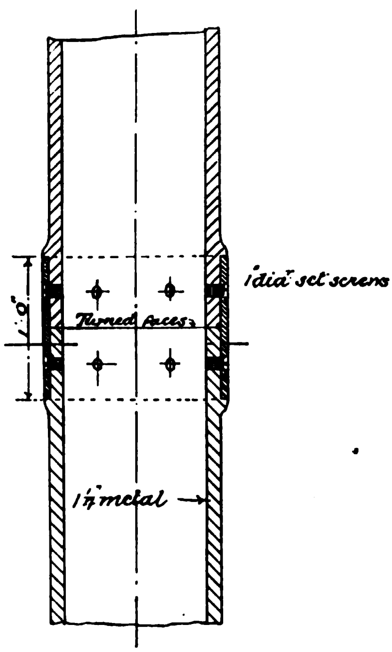


FIG. 353.
Scale ¾ inch = 1 foot.

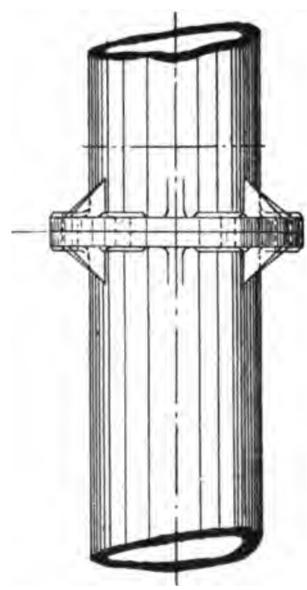


FIG. 354.
Scale ¾ inch = 1 foot.

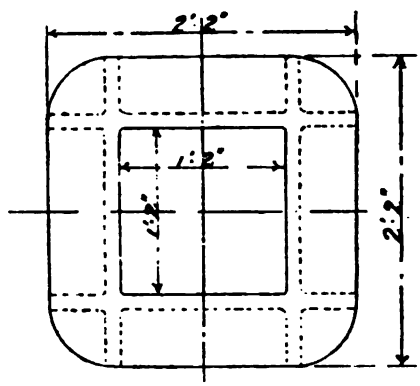
girders, of which the end of one next the main girder is seen in Fig. 360, carry the decking, which consists of trough flooring plates, as shown, covered with asphalte, and supporting ballast filling upon which the sleepers of sundry lines and crossings are laid, these latter serving the purpose of railway traffic between the jetty and the mainland.

As the main girder flanges are 3 feet in width, while the cylinder below is 7 feet in diameter with a capping 8 feet diameter, the curb or coping line of the jetty is brought forward, so as to be flush with the cylinder cap below, in the manner shown in Fig. 360, riveted steel brackets being attached to the box girder at intervals, supporting a capping of heavy timbers, as shown.

The box girders are of dimensions sufficient to enable painting to be done inside, and means of access are provided.

The attachment of a cross girder to the main box girder in rear of the latter is shown in Fig. 361.

The attachment of the bollards in this structure is a detail of



PLAN ON CAP.

FIG. 355.
Scale ¾ inch = 1 foot.

importance, and is of more complex type than the single concrete foundation which can be employed under other conditions. It is shown in Figs. 359, 362, and 363, the bollard, of special section,

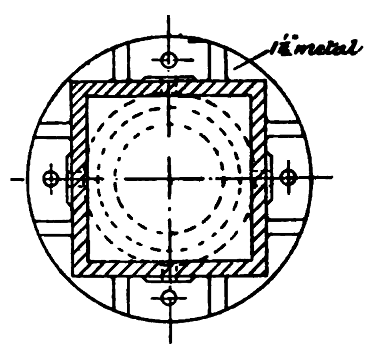


FIG. 356.
Scale ¾ inch = 1 foot.

being inserted between the webs of main box girders, as shown in Fig. 363, and carried down below the seatings of those girders *into* the concrete filling of the cylinder below, the pull on the bollard

being further resisted by diagonal ties carried back to the wall behind, the whole being intended to resist the pull of hawser of a heavy vessel.

The cylinders are of massive construction, and are spaced, as

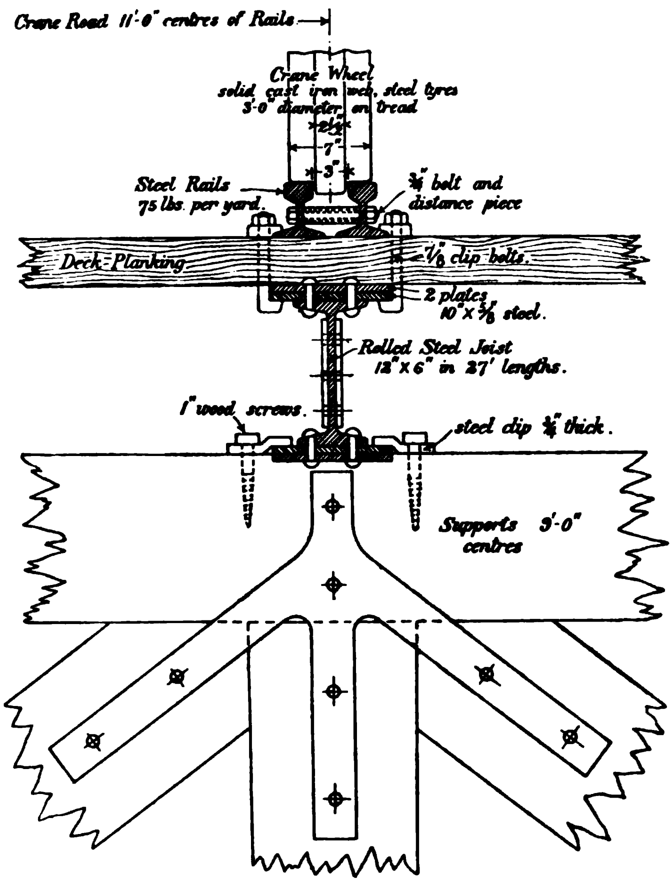


FIG. 357.
Scale ¾ inch = 1 foot.

before stated, about 60 feet apart, longitudinally, in a single row. The upper portion of the cylinder, 7 feet in diameter, is shown in Fig. 358, cast in complete rings, 1½ inch thick, and connected by flanged joints with 1¼-inch bolts through drilled holes. Cast holes are frequently used in this class of work, but holes drilled to a templet are far more satisfactory, and secure complete

interchangeability of parts. The advantages gained in this respect in erection counterbalance the additional cost. The meeting surfaces of the flanges are machined all over, it being virtually no more costly than the machining of separate strips. Watertightness during the process of construction and sinking is thus attained, assisted by the use of canvas and red lead jointing material, or by

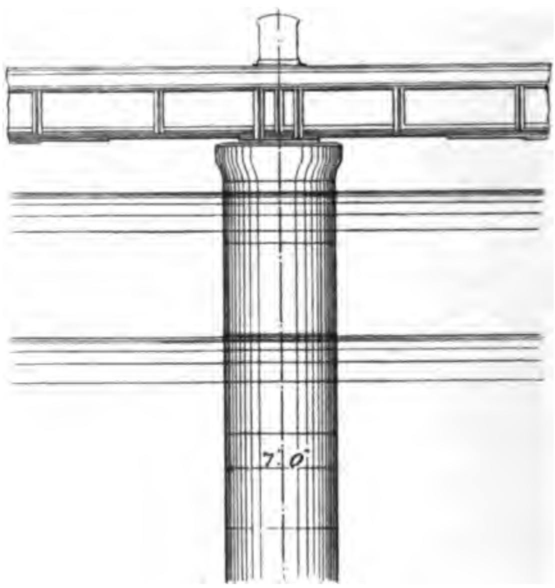


Fig. 358.
Scale 1 inch = 12 feet.

the insertion of india-rubber cord flattened out by the squeeze of the bolts.

Details of the cap to the column are shown in Fig. 364 in section, and in plan in Fig. 365.

The outline of the cap is designed with simple and easy curves, as shown, without projecting mouldings, in order that small floating craft, such as barges, etc., may not catch their coamings or fenders on a rising tide, the cap being but a short distance above high water.

The make-up length, to allow of deviations from regularity in the final level to which the cylinders are sunk, is the one immediately below the cap, as shown in Fig. 358.

The lower portion of the cylinder is shown in Fig. 366. The three lowermost rings are 10 feet in diameter, 1½-inch metal, being

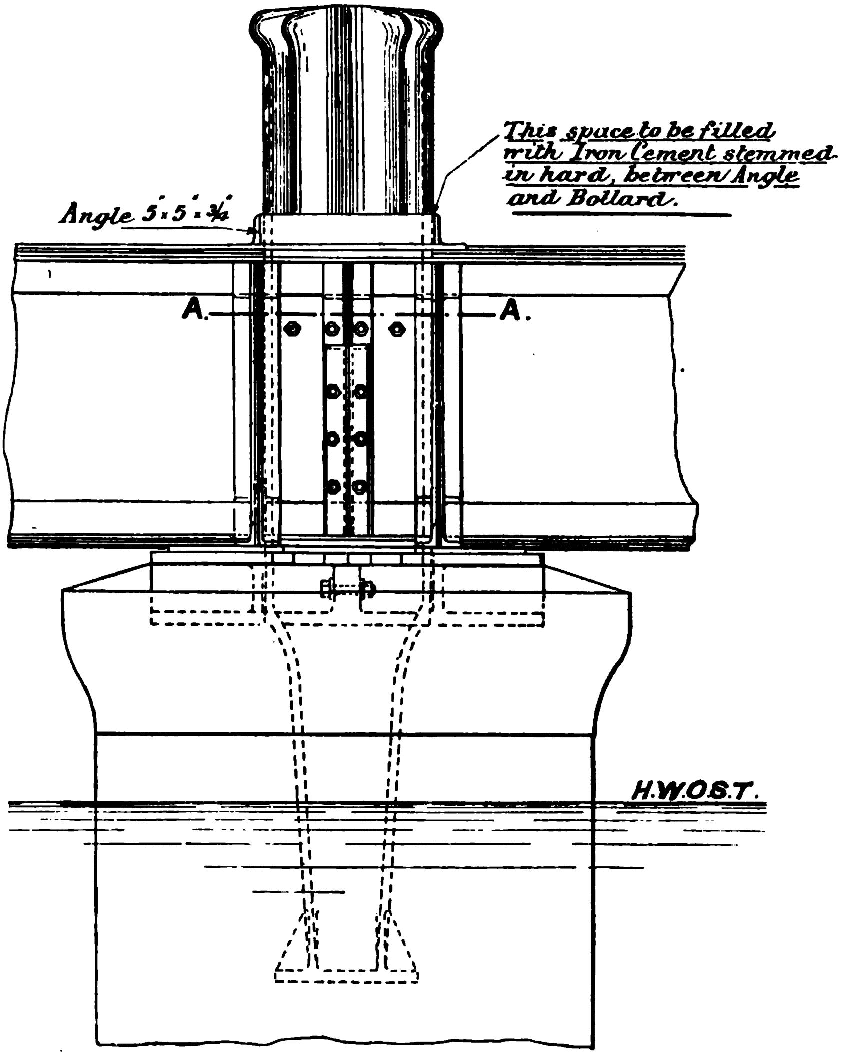


FIG. 359.
Scale ⅜ inch = 1 foot.

enlarged to give greater area for bearing, and working space for excavation and sinking operations. The connection between the

lower rings of 10 feet diameter and the upper rings of 7 feet is made by the conical length shown in Fig. 366.

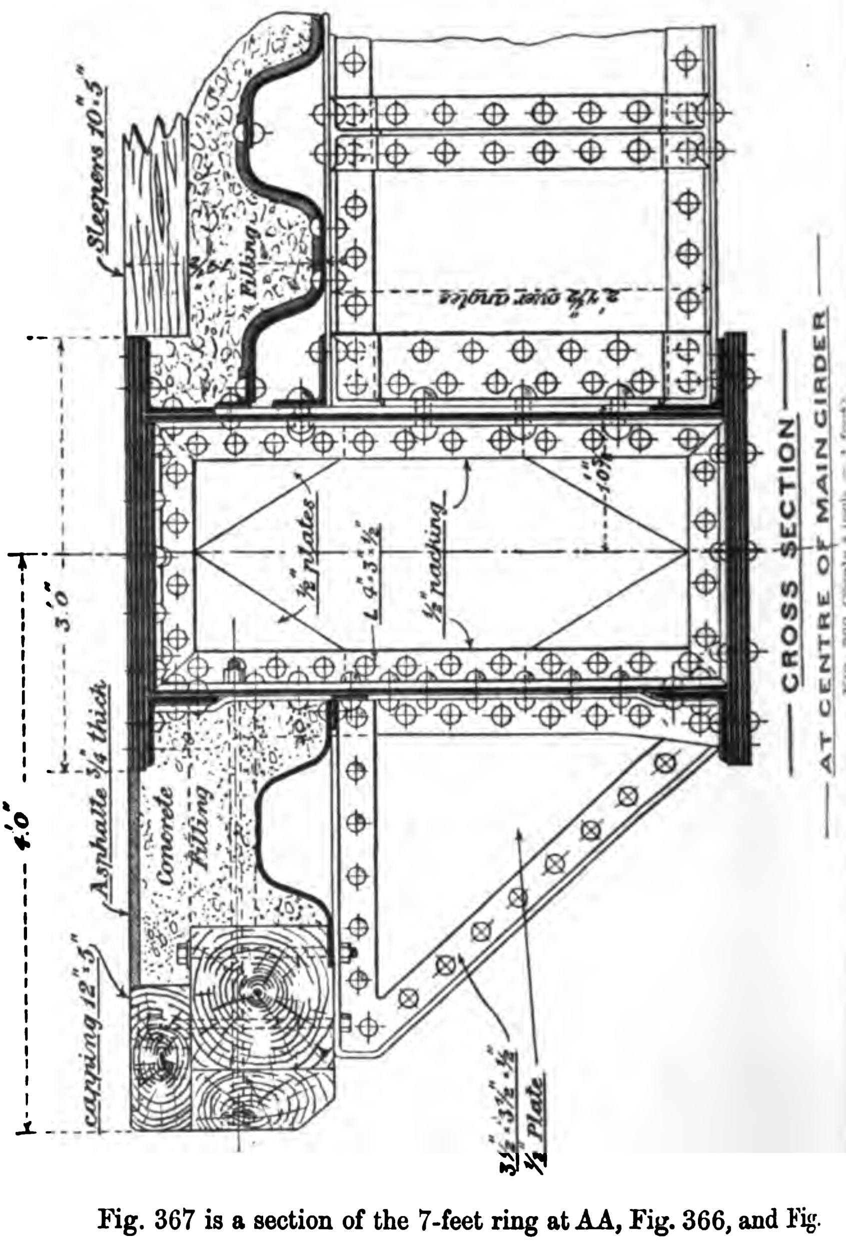


Fig. 367 is a section of the 7-feet ring at AA, Fig. 366, and Fig.

368 is a section at BB, looking up, and showing the interior of the conical length.

The cutting edge of the lowermost length of 10-feet cylinders is shown in detail in Fig. 35, the bottom portion, 8 inches in depth, being thickened out to 2 inches, strengthening the cutting edge.

A further use of iron or steel in the construction of marine

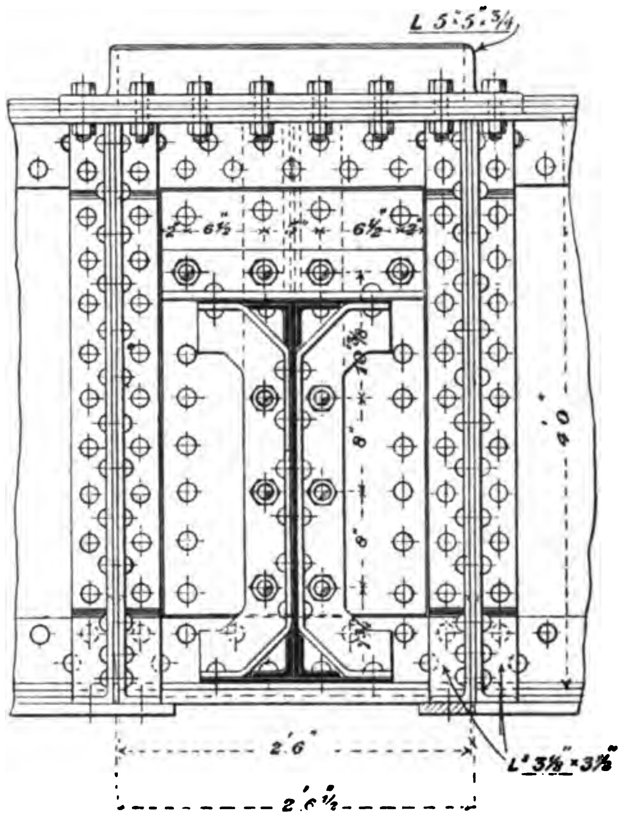


FIG. 361.
Scale ¾ inch = 1 foot.

works in wharf or dock walls may be found in the construction of heavy tie-rods, which are occasionally found necessary to prevent forward movement of walls due either to weakness of foundations, or heavy surcharged loads (the wharf wall being considered as a retaining wall), such as coal stacks, etc.

An example of such tie-rods will be found in Figs. 369 to 376 inclusive, which represent the details of a tie-rod for anchoring back the summit of a wharf wall to a rubble mound considerably in the rear of the wall. The rod is $2\frac{1}{4}$ inches diameter in the main section, of mild steel, jointed as shown in Figs. 369 and 370, which show the details of the coupling boxes. The rods are swelled at the ends by means of an hydraulic forging press, and "plus" threaded. The coupling boxes are shaped as shown, while the material of which they are made is described in the results of

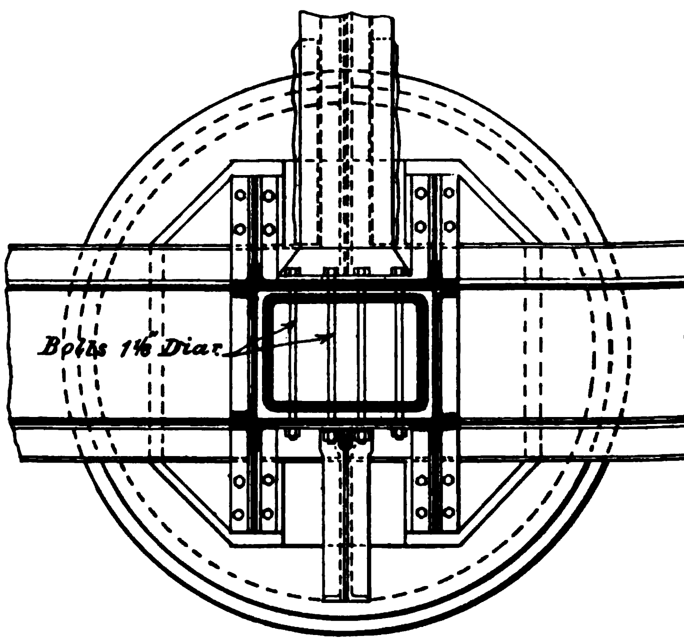


FIG. 362.
Scale ⅜ inch = 1 foot.

the tests given on p. 45. The detail of the attachment of the tie-rod to the wall is given in Fig. 371, a recess being cut or left in the face of the wall, into which the cast-iron washer, shown in the figure, is inserted, the whole being flushed up with fine Portland cement concrete, thus preserving the face of the wall intact and preventing obstructions.

The anchorage of the other end of the rod is shown in Fig. 372, where the tie-rod is shown forked into two 2-inch diameter branches, taking hold of a concrete block of such dimensions as to

prevent its being dragged through the rubble in which it is embedded.

The junction of the three rods is shown in Figs. 373, 374, and the anchor plates in Figs. 375, 376.

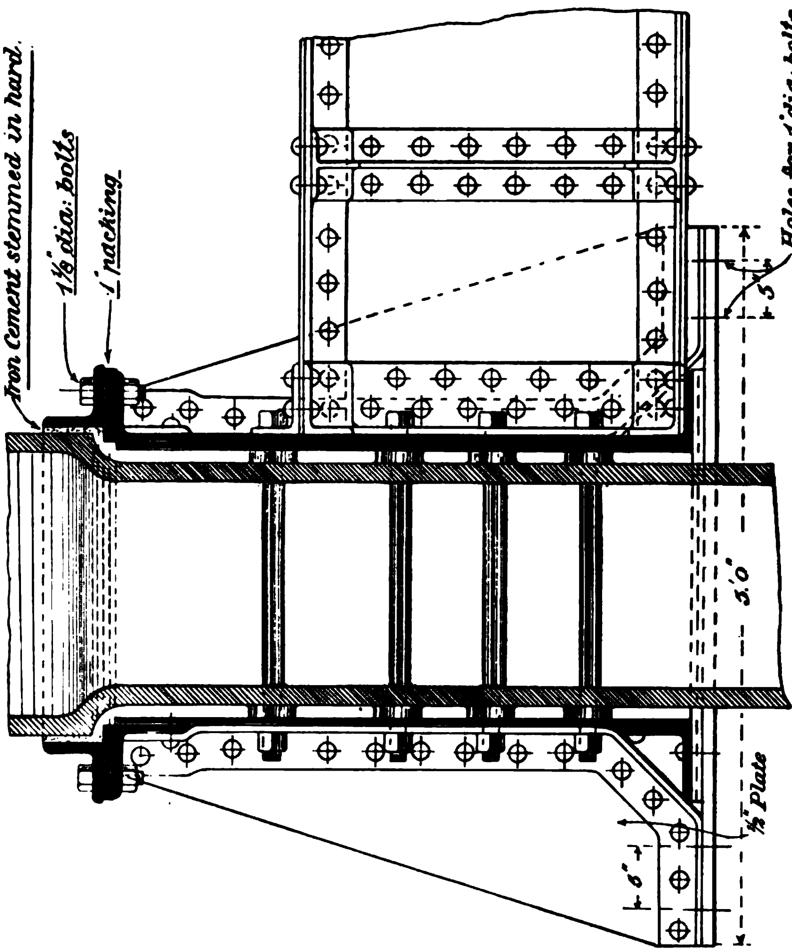


FIG. 363 (Scale ¾ inch = 1 foot).

The whole of the tie-rod is embedded in a rough pine-wood box, and rammed round with a mixture of sand and pitch to preserve from corrosion, the rod being buried in the filling some feet below the surface.

The use of the concrete anchorage as described above is an exceptional case, the tie-rods being used mainly in the tying

together of two parallel wharf walls, the rod passing through from the face of one wall to the face of the other.

FIG. 364 (Scale ⅜ inch = 1 foot).

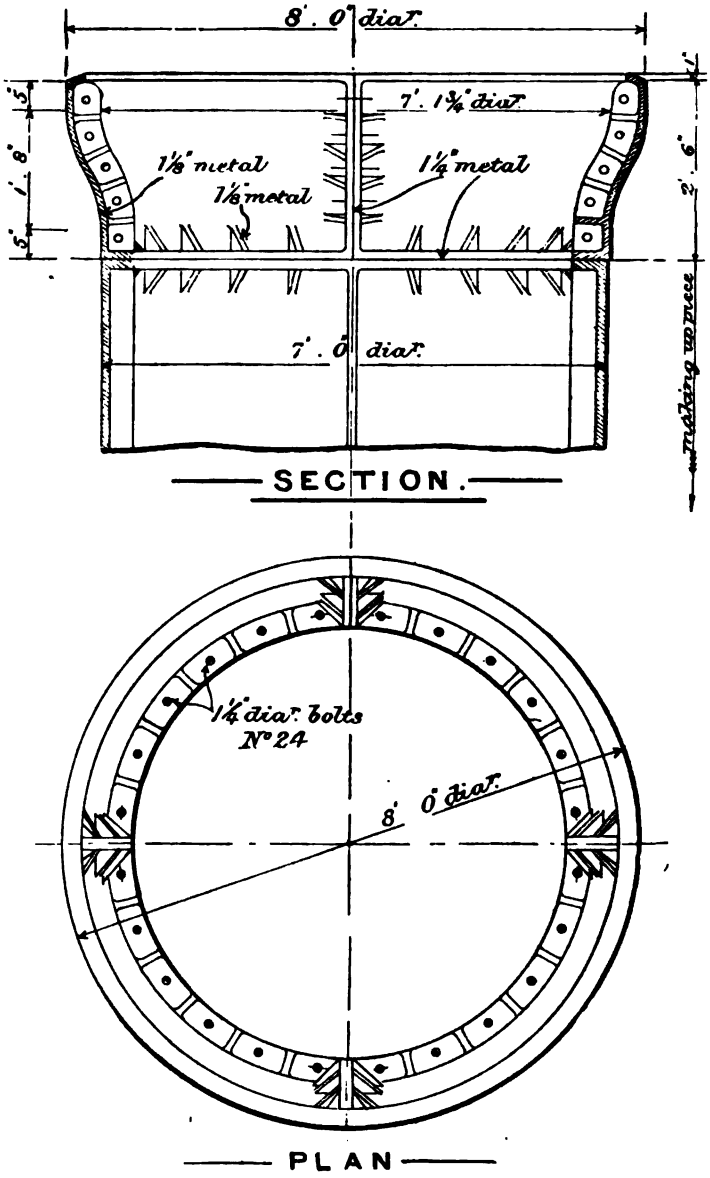


FIG. 365 (Scale ⅜ inch = 1 foot).

Bollards.—Among the necessary items of equipment on a wharf, jetty, or dockside, for the handling of ships or vessels of all

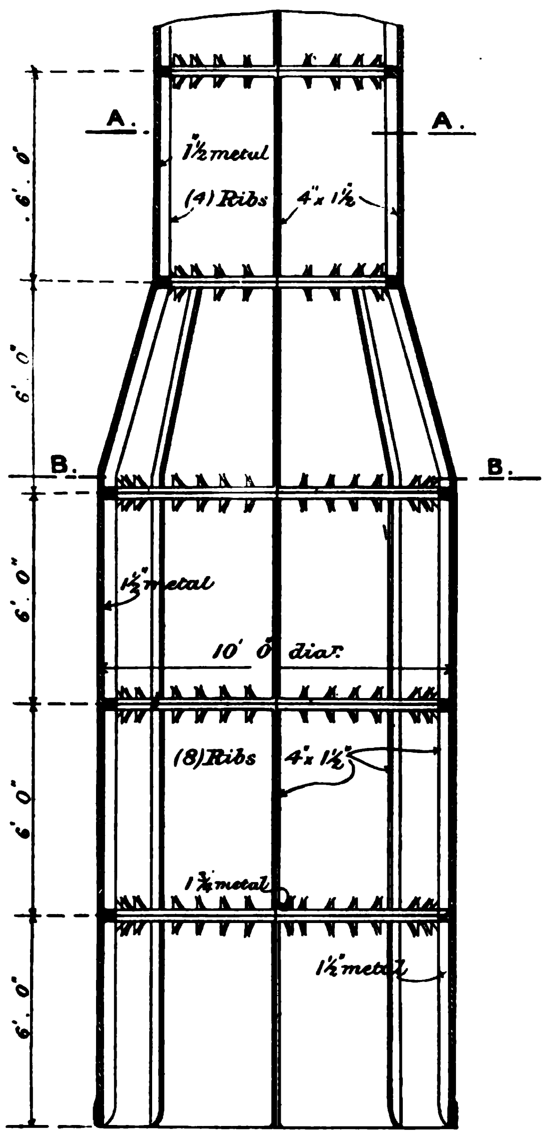


FIG. 366.
Scale $\frac{3}{16}$ inch = 1 foot.

sizes, not the least important are the bollards for the attachment of mooring hawsers.

These may be occasionally of stone, and are frequently of timber in those cases where the outer piles of a timber jetty of pier are carried up above the level of the deck. In modern dock work, where the class of craft which comes alongside includes some of the heaviest ships afloat, it is customary to make the bollards of

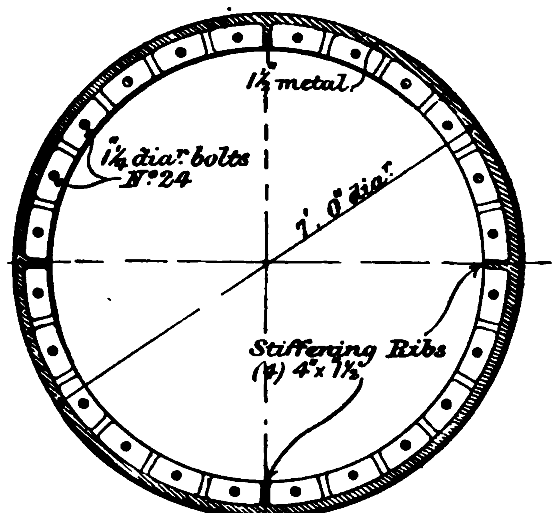


FIG. 367.
Scale $\frac{3}{8}$ inch = 1 foot.

cast iron, and latterly in first-class work they have been constructed of cast steel.

It is obvious that where very large vessels are concerned it is necessary for safety in handling that the detail of attachment of the shore end of the hawser should be above suspicion as far as it is possible to make it so, as the failure of a bollard at a critical moment may have serious results. The consideration of the stresses which may arise from the pull of a steel wire or hempen hawser will lead to the conclusion that the case is not quite so simple as may appear at first sight.

The direction of the pull of the hawser in a horizontal plane may be at any angle within a semicircle on the wharf side, or even practically all round the circle in those cases where a lead can be taken off a bollard in almost any direction, while there will

frequently exist a vertical component of pull tending to pull the bollard out of the ground in cases of attachment to a high-sided ship.

Again, there may exist a powerful element of torsion whenever the hawser, as is frequently the case, is taken with several

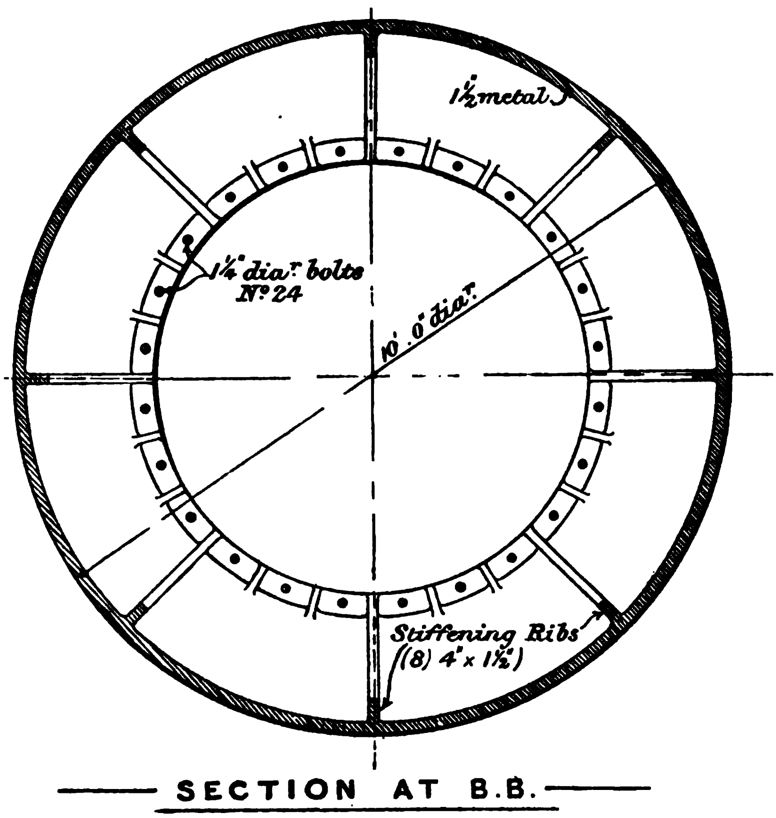
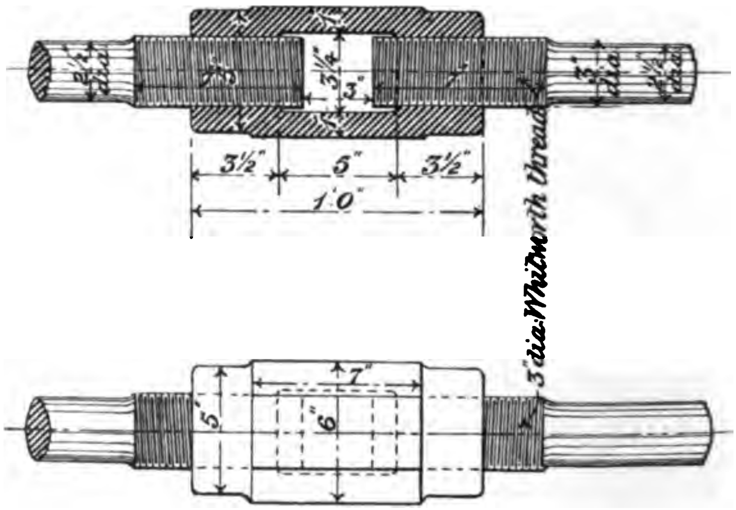


FIG. 368.
Scale ⅜ inch = 1 foot.

turns round the bollard, and the pull is at one side or edge instead of being led to the centre.

In all the varying degrees of angle at which the pull may occur, the ultimate strength of the hawser itself may, however, be taken as the measure of the strength of the bollard. When the hawser parts, the duty of the bollard is at an end, while it is

probable that the strength of the shore attachment, as usually designed, is at least equal to that customary on board ship. The



Figs. 369, 370.
Scale 1¼ inch = 1 foot.

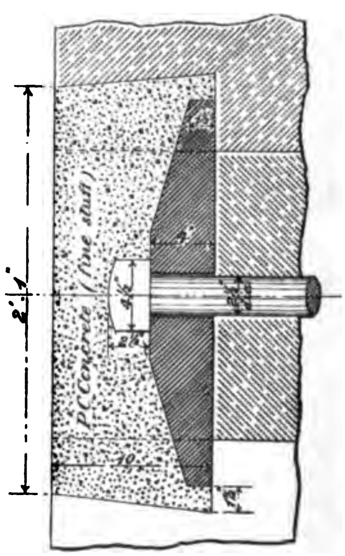


Fig. 371.
Scale 1 inch = 1 foot.

consideration of this latter detail belongs, however, to the domain of naval architecture.

The calculation of bollard strength on these lines presents no

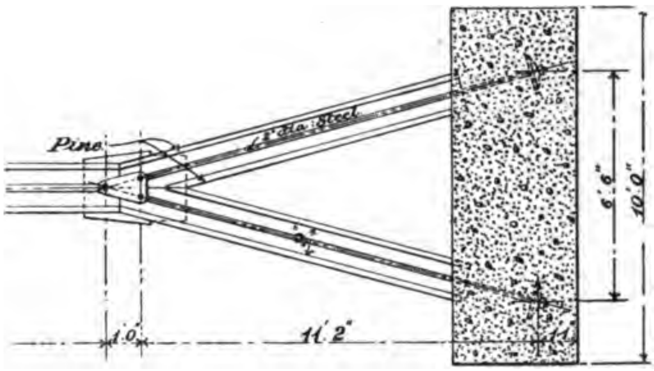


Fig. 372.
Scale $\frac{3}{16}$ inch = 1 foot.

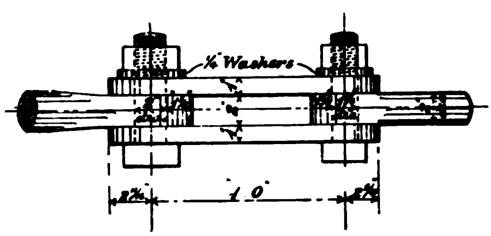


Fig. 373.
Scale 1 inch = 1 foot.

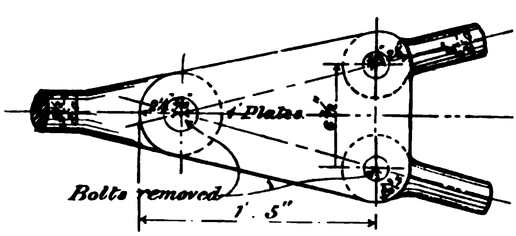


Fig. 374.
Scale 1 inch = 1 foot.

special difficulty, using the moment of inertia of the section under consideration, and assuming the maximum height or lever arm at which the pull is likely to take place, while the torque can be

calculated in the usual manner. The strength of the foundation in which the bollard is fixed must necessarily be governed by

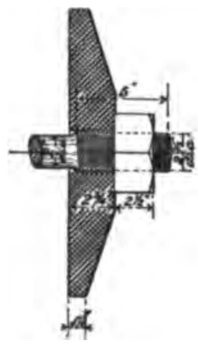


FIG. 375.
Scale 1 inch = 1 foot.

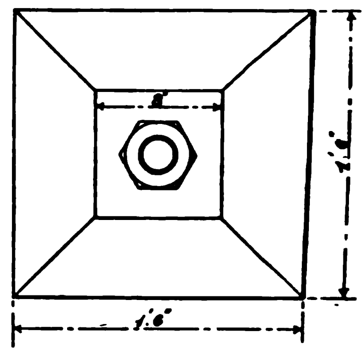


FIG. 376.
Scale 1 inch = 1 foot.

circumstances, and is not always capable of precise calculation. The bollard may be fixed in a solid wall, and there may be cases

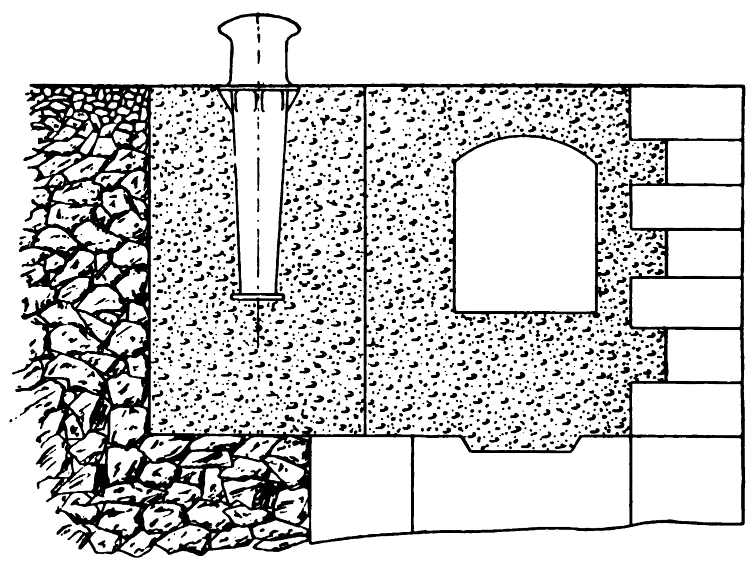


FIG. 377.
Scale $\frac{1}{16}$ inch = 1 foot.

in which the stability of the whole wall must be considered, while if fixed near the edge, the resistance to bursting of the masonry or

concrete in front of the bollard must not be lost sight of, and where this resistance is insufficient or doubtful, it has to be supplemented by ties to the back of the wall. In those cases where a bollard is fixed in an isolated block of concrete, which itself is buried in soil, possibly made ground, the resistance to overturning of the concrete block is supplemented to some extent by the resistance of soil of varying degrees of softness or compressibility, and

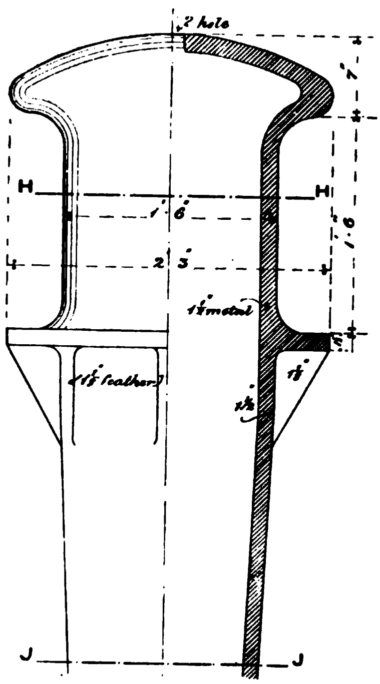
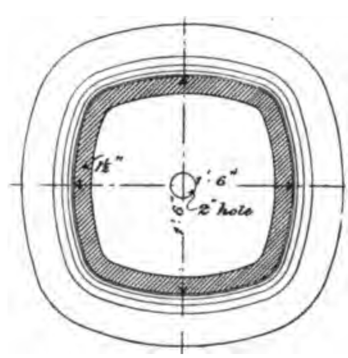


FIG. 378.
Scale ¾ inch = 1 foot.



SECTIONAL PLAN.
ON LINE H-H.
LOOKING UP.

FIG. 379.
Scale ¾ inch = 1 foot.

in such cases the total resistance offered may not be easily calculable. It is certainly possible, and has occurred in practice, that the bollard with its foundation may be pulled out of perpendicular, aided by a vertical component of stress, before the transverse resistance of the bollard itself had been reached. This part of the subject belongs, however, to the theory of the strength of foundations generally, or rather to that branch of it which deals with resistances in soils other than vertical.

The position of bollards with relation to the edge of coping or

wharf line is often determined by varying considerations. In some cases it is deemed desirable to keep the bollards back and leave a working space alongside the edge of jetty or dock, but in these cases the space in question will be encumbered by the hawsers themselves. Where this is deemed objectionable, as in those cases where a railway line, or crane road of wide gauge, or both combined, are required alongside the wharf or dock, then the

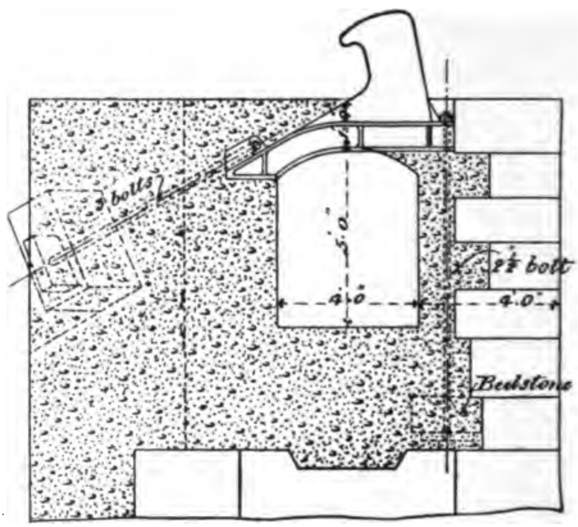


FIG. 380.
Scale $\frac{1}{8}$ inch = 1 foot.

bollard is frequently placed as close to the edge as is practicable, and in many cases is placed flush with the face of the wharf wall.

These varying conditions give rise to a variety of detail governed by the various methods employed of tieing back the bollard to a sufficiently good foundation to meet all the conditions of stress.

In Fig. 377 we have the case of a simple form of bollard embedded in a concrete block at the back of a quay wall, there being a subway in the top of the wall to contain hydraulic or electric mains, water pipes, etc. The upper portion of this bollard is shown in part elevation and section in Fig. 378, and in sectional plan on the line HH in Fig. 379, which shows the shape of that portion of the bollard to which the hawser is attached, the lower portion, embedded in the concrete foundation, being made square

to prevent turning round under torsional stresses. It will be observed that the lower portion of the square is cast with a projecting lip offering resistance to a vertical component in the pull. The mushroom-shaped head at the top of the bollard prevents slipping upwards of the hawser.

A special form of cast-iron bollard to meet special conditions is shown in Fig. 380. In this case it is required to place a bollard as near the coping line of a commercial wharf as possible, in order to leave behind it an unencumbered space for crane and wharf roads. The outer rail of the crane road is placed on the coping to

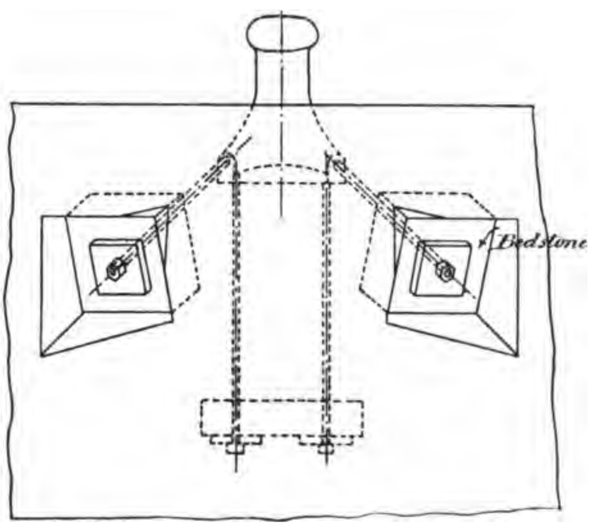


Fig. 381.
Scale $\frac{3}{16}$ inch = 1 foot.

facilitate the design of the travelling crane to be employed and reduce overhang. The space required for this rail (not shown in the figure), and the dimensions of the crane leg and its gear, require the bollard to be set back a little from the coping line as shown, while we have almost immediately under the bollard a subway for the purposes above described.

This combination of conditions gives rise to the form shown in Fig. 380. The under surface of the casting is shaped as shown to suit the outline of the subway, while the necessary tieing back is secured by the 3-inch diameter bolts which are carried to the back of the wharf wall, which is widened out in way of the bollards to provide sufficient mass.

Fig. 381 is an elevation of the rear of the wall showing the attachment of the tie-rods, and Fig. 382 is a plan of the general arrangement. Fig. 383 is a longitudinal section of the front portion of the bollard, and Fig. 384 is a section through MM, Fig. 383.

Figs. 385, 386, and 387 are details of the rear portion of the bollard, showing the attachments of the 3-inch diameter ties. These ties pass through a cast-iron washer similar to that shown

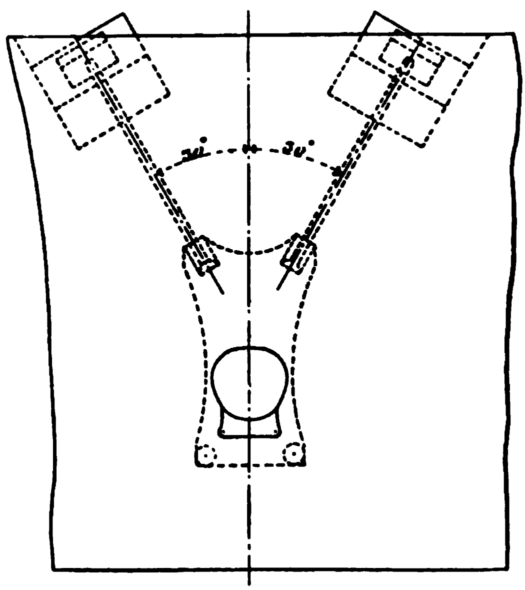


FIG. 382.
Scale $\frac{1}{8}$ inch = 1 foot.

in Fig. 375, resting on a stone bed plate embedded in the concrete of the wall.

The front portion of the bollard is held down by a pair of $2\frac{1}{2}$-inch diameter rods, embedded in the wall as shown.

Another form of bollard suitable for heavy vessels, and adopted in its details for attachment to a jetty of composite construction having a timber superstructure, is shown in Fig. 346, while the attachments of a bollard of special construction to suit the details of riveted steel girderwork in a jetty of another type are shown in Figs. 358, 359, 362, 363.

In combination with a system of bollards, the complete equipment of a modern jetty or wharf will also include fairleads

and capstans, the latter being driven by steam, compressed air, hydraulic, or electric power, in accordance with the type of power available.

The relative position of bollards, fairleads, and capstans should be carefully studied and planned out in order that the lead of the hawser may be as free as possible and not exposed to chafing.

Where very heavy vessels have to be manipulated, the

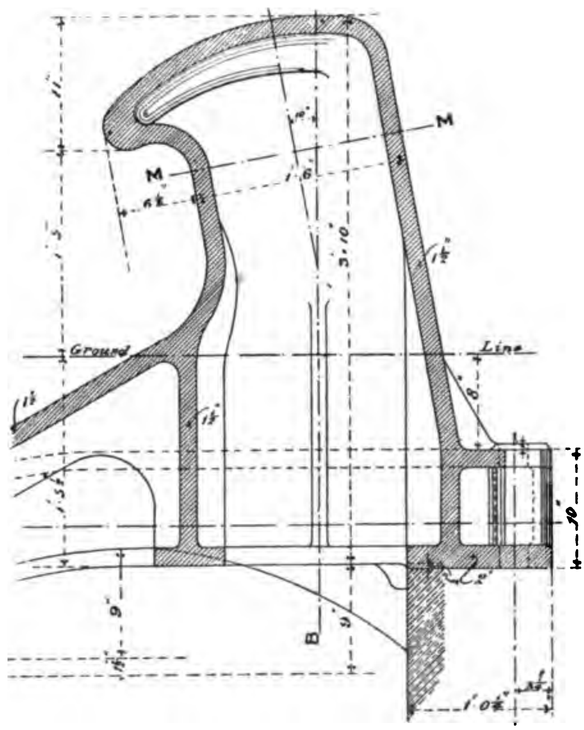
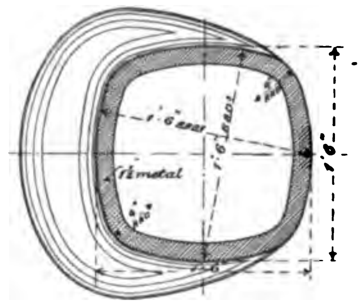


Fig. 383.
Scale ⅜ inch = 1 foot.

anchorage of the fairleads should be of a substantial character, and it is desirable that the foundation should offer ample resistance to drawing out or overturning. If a fairlead be planted, for example, on a coping stone, damage by loosening the stone may result, unless the latter be very firmly secured. An independent foundation, consisting of a concrete block of sufficient dimensions, is often found a preferable course.

The general arrangements of bollards, fairleads, and capstans

at the entrances to docks and basins will generally require a careful study in detail, not only to secure the best and most advantageous positions for the handling of incoming and outgoing vessels, but also that they may occupy positions in proper relation

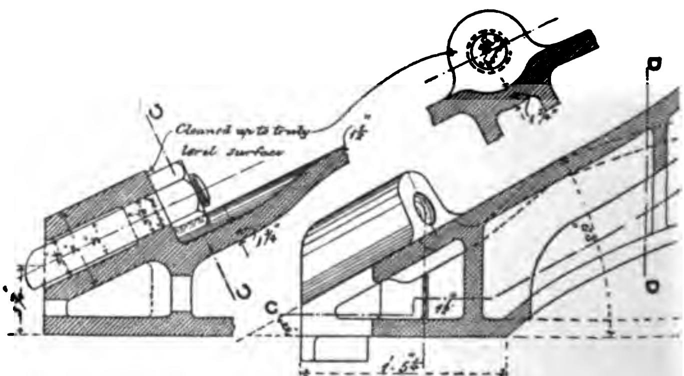


— SECTION ON LINE M-M. —

FIG. 384.*
Scale ¾ inch = 1 foot.

to each other, and not interfere with various machinery details which commonly occur in such situations, such as penstock or caisson machinery of various kinds.

Caissons.—The term "caisson" is applied to a variety of



FIGS. 385, 386, 387.
Scale ¾ inch = 1 foot.

structures fulfilling varied functions and serving widely different purposes. Thus we have the term applied to that form of structure

designed to close dock entrances, and forming virtually a movable dam. Reference to this form of construction will be made later on. Or we may find it applied to those appliances by the aid of which the foundations of important structures, such as large bridges, are sunk to great depths, often under conditions of great difficulty and risk. Or again, as before mentioned, the term is applied to a class of structure by means of which breakwaters constructed in deep water are either commenced or, as it sometimes happens, are brought to a conclusion by the formation of the head of the breakwater inside, or by means of, a caisson.

In this chapter practical examples, recently carried out, will be given of two classes of caissons, viz. those employed under special circumstances for the commencement of an important breakwater, and those employed for the closing of dock entrances.

A recently completed breakwater,[1] constructed of concrete blocks, laid by what is known as the sloping system of blockwork was commenced subsequently to the deposit of an extensive rubble mound, designed so as to bring the foundation level of the blockwork from the sea bottom upwards to about 36 feet below low water, by a steel caisson designed by the author, and of the form shown in longitudinal section in Fig. 388. This caisson may be described as a rectangular box with sloping ends, and having the following principal dimensions:—

Length moulded on bottom	101′ 2½″
Length moulded on top of fixed part ...	73′ 10½″
Length moulded on portable bulwarks ...	72′ 10½″
Breadth moulded	33′ 0″
Depth moulded of fixed part	37′ 6″
Height of portable bulwarks	11′ 0″
Length of well moulded	60′ 0″
Angle of sloping ends	70°

The caisson was of a uniform moulded breadth, as above, of 33 feet, this being the normal width of the blockwork construction, while the slope of the end portions, 70 degrees, was the angle at which the sloping blocks were laid, both ends of the caisson being sloped to the same angle, the intention being to sink the caisson at or near the centre of the breakwater and commence work in

[1] For a description of this work, the student is referred to the article on "Dockyards" by the author of these notes in the *Supplement to the Encyclopædia Britannica.*

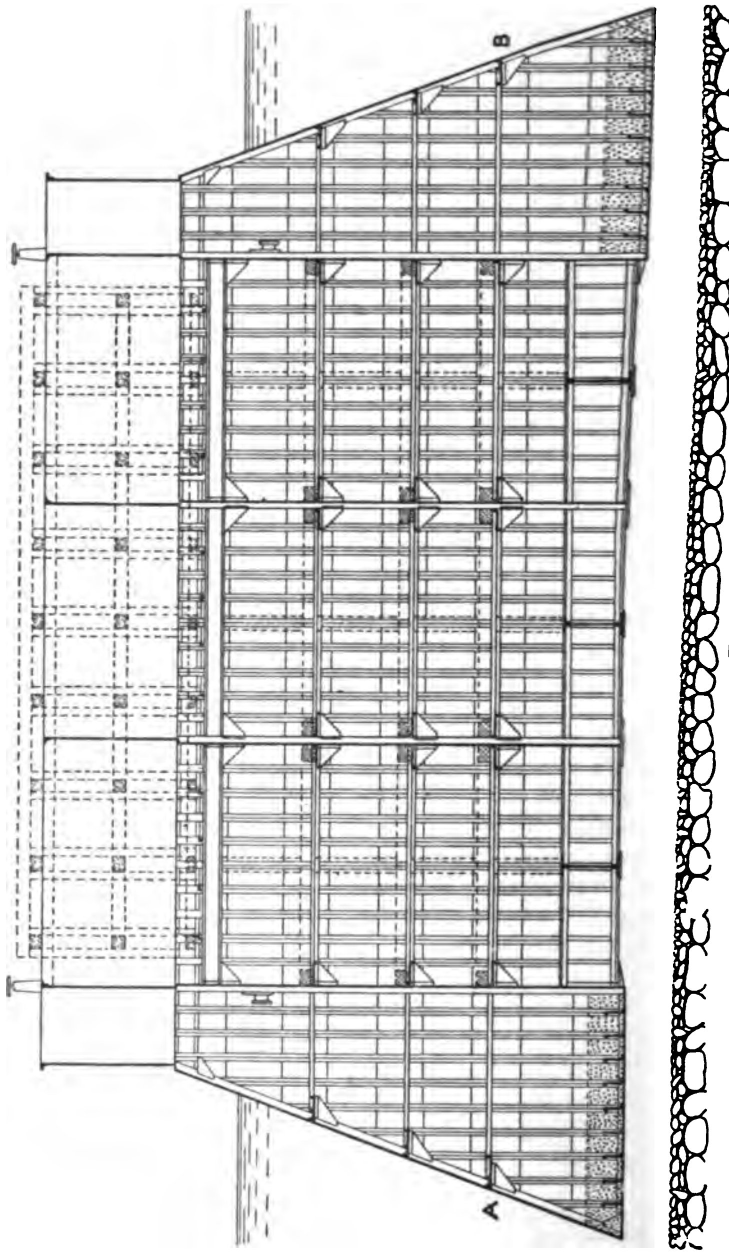


FIG. 388.
Scale 1 inch = 16 feet.

both directions simultaneously, the blockwork in one-half of the breakwater sloping in the opposite direction to that in the other half.

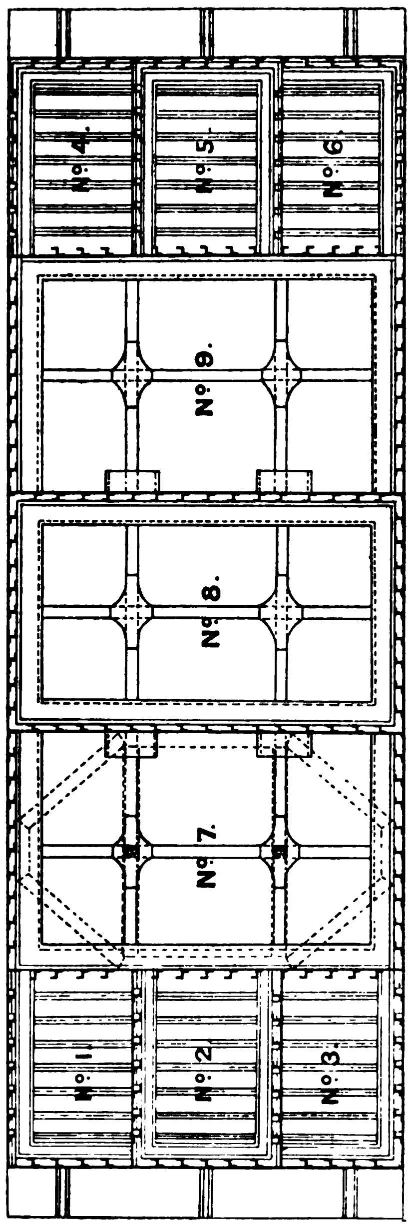


FIG. 389.
Scale 1 inch = 16 feet.

The caisson was divided lengthwise into five chambers, the two end chambers being watertight compartments with a combined total displacement calculated to ensure the caisson, with all ballast and temporary fittings and gear on board, floating with a maximum draught of 32 feet, thus having a clearance of 4 feet under the bottom, over the finished and prepared surface of the rubble mound before mentioned.

The three central compartments formed an open well, with no bottom, and having a total moulded length of 60 feet, each compartment being 20 feet in moulded length, with a breadth moulded of 33 feet.

The intermediate bulkheads and the side plating of the central well were watertight, the bottom being open, but framed with strong plate girders longitudinally and transversely, as shown in the figures.

The end watertight compartments were each subdivided by fore-and-aft bulkheads into three separate watertight compartments, the entire structure being thus divided into nine compartments, as shown in plan in Fig. 389 (which is a section on A, B, Fig. 388) and numbered 1 to 9. The object of this subdivision will be explained later on. Fig. 390 is a cross-section of the caisson taken partly through the end compartments, and partly through the central well.

The caisson was constructed of mild steel of the usual quality.

The frames were of 8″ × $3\frac{1}{2}$″ × $3\frac{1}{2}$″ × $\frac{3}{8}$″ Zeds, spaced 2 feet apart, centres, the stringers in the end watertight compartments, Nos. 1 to 6, consisting of a web plate 24″ × $\frac{3}{8}$″, cut for the frames, and brought up to and riveted to the skin plating in the usual way, with an angle steel 3″ × 3″ × $\frac{3}{8}$″ for each flange.

The stringers to compartments 7, 8, and 9 consisted of a web plate 24″ × $\frac{3}{8}$″, with flange angles 3″ × 3″ × $\frac{3}{8}$″, attached to the bulkheads and frames by means of brackets and bolted connections in such a way as to render them removable in the central well compartments prior to the deposit of concrete, this course being adopted in order that the deposited concrete in the central well might have as few dividing planes of weakness as possible. The stringers in compartments Nos. 1, 2, 3, 4, 5, and 6 were not removable, being riveted to the skin plating.

The skin plating was of mild steel plates, arranged in and out, nine strakes in height, the three lowermost being $\frac{1}{2}$ inch thick, the three next $\frac{7}{16}$ inch, and the three uppermost $\frac{3}{8}$ inch thick. The

landings of skin plating and bulkheads were double riveted throughout, with double riveted butt straps, and double riveted overlap butts to bulkheads.

The riveting, principally $\frac{3}{4}$ inch diameter, was spaced 4 to $4\frac{1}{2}$ diameters in watertight work, 7 to 8 diameters in non-watertight work, and with special spacings elsewhere.

The "floors" or floor girders stiffening the bottom $\frac{1}{2}$-inch

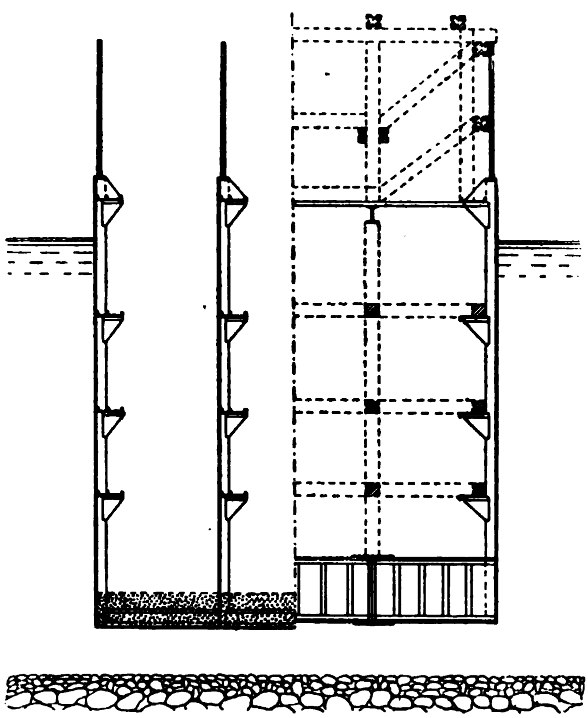


FIG. 390.
Scale 1 inch = 16 feet.

plating of end watertight compartments were of 18″ × $\frac{3}{8}$″ web, with one 3″ × 3″ × $\frac{3}{8}$″ angle to each flange.

The plate girders at the bottom of the central well compartments Nos. 7, 8, and 9 were longitudinal and intercostal, as shown, and consisted of webs 60″ × $\frac{3}{8}$″, with top and bottom flanges of two 6″ × 6″ × $\frac{1}{2}$ angles, double riveted, with one plate $12\frac{1}{2}$″ × $\frac{1}{2}$″, the webs being stiffened at 2 feet intervals with 3″ × 3″ × $\frac{3}{8}$″ angles, the junctions of longitudinal and intercostal girder being

further stiffened by top and bottom sketch plates as shown. The purpose of these stiff bottom girders was to resist, in combination with concrete, upward water pressure at certain stages of the filling of the central well with concrete, and pumping out of the several compartments.

The stringers supporting the frames in the central well, compartments 7, 8, and 9, were not, for economical reasons, designed to resist the full collapsing pressure of the water which came upon them in process of pumping out of the compartments, and they were accordingly strengthened by temporary timber struts of pitch pine, 14 inches square, arranged as shown in plan, Fig. 389,

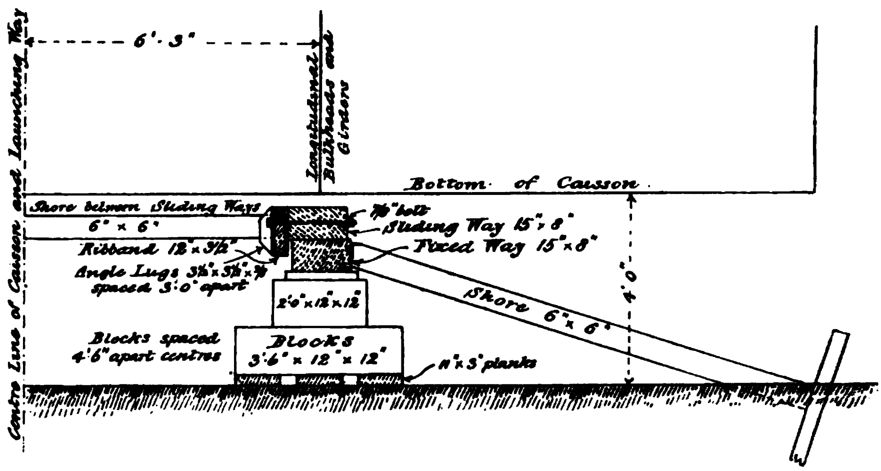


FIG. 391.
Scale ¼ inch = 1 foot.

and arranged in tiers as shown in Figs. 388 and 390. These struts were removed as the concreting of the structure was brought up.

The upper portion of the caisson was surmounted by portable steel bulwarks of $\frac{5}{16}$-inch plate stiffened with angles and strutted internally with timber, as shown in Figs. 388 and 390, this strutting being so arranged as to form a platform on top for the temporary work connected with the flotation, sinking, and concreting of the structure.

Six sluice valves, 12 inches in diameter, lined with gun metal and tested to 150 lbs. pressure, were supplied, one to each subdivision of the end watertight compartments, Nos. 1, 2, 3, 4, 5, and 6.

These valves were worked, as shown, from the upper platform,

and were for the purpose of admitting water to the end compartments aforesaid.

Mooring rings were attached to the caisson at convenient places, and the sloping ends were provided outside the skin plating with angle irons riveted to the plating and running vertically up the slope, to form a key with the first layer of concrete blocks laid against the slope, and moulded with corresponding grooves in the blocks, to minimise the risk of the first row or rows of blocks being shifted off the face of the caisson by sea stroke until they received the full weight of the succeeding tiers.

The caisson was constructed and put together in the building-yard at home, and was subsequently taken to pieces and shipped to the site of the works, there to be re-erected, riveted, and launched, completed in the water, floated out to the site of the breakwater, and sunk.

Local conditions connected with the site of launching and depth of water available, made it desirable that the launching draughts should not exceed 2 feet $1\frac{1}{2}$ inch forward, and 4 feet $6\frac{1}{2}$ inches aft, and to obtain these conditions the caisson was launched in an incomplete state, with only so much of the framework and skin plating erected as was compatible with the above conditions.

The caisson was launched end on, in preference to broadside on, the bottom of the central well compartments being temporarily decked over with 4-inch planking of sufficient watertightness to serve for launching purposes. This was subsequently removed when the caisson had been towed into sufficiently deep water.

A section of the launching ways is shown in Fig. 391, the ways being laid with a declivity commencing with $\frac{9}{16}$ inch per foot, and terminating with $\frac{15}{16}$ inch per foot at low water, the declivity of the caisson itself being $\frac{9}{16}$ inch per foot.

A minimum depth of water of 11 feet at low water, extending outwards for about 60 feet beyond the end of the ways, with a width of 70 feet, sufficed for launching purposes, the launch being successfully effected.

The subsequent process of erection and completion of the caisson in the water was but a repetition of the process of building up successive additions of steel-work in framing and skin plating, and the gradual loading up of ballast, until the entire structure, including all temporary timber work and other temporary appliances, was completed, and sunk to its final draught on an even keel of 32 feet, as designed.

In view of the sea risks to be encountered during all the stages subsequent to the launch, of completion in the water, floating out

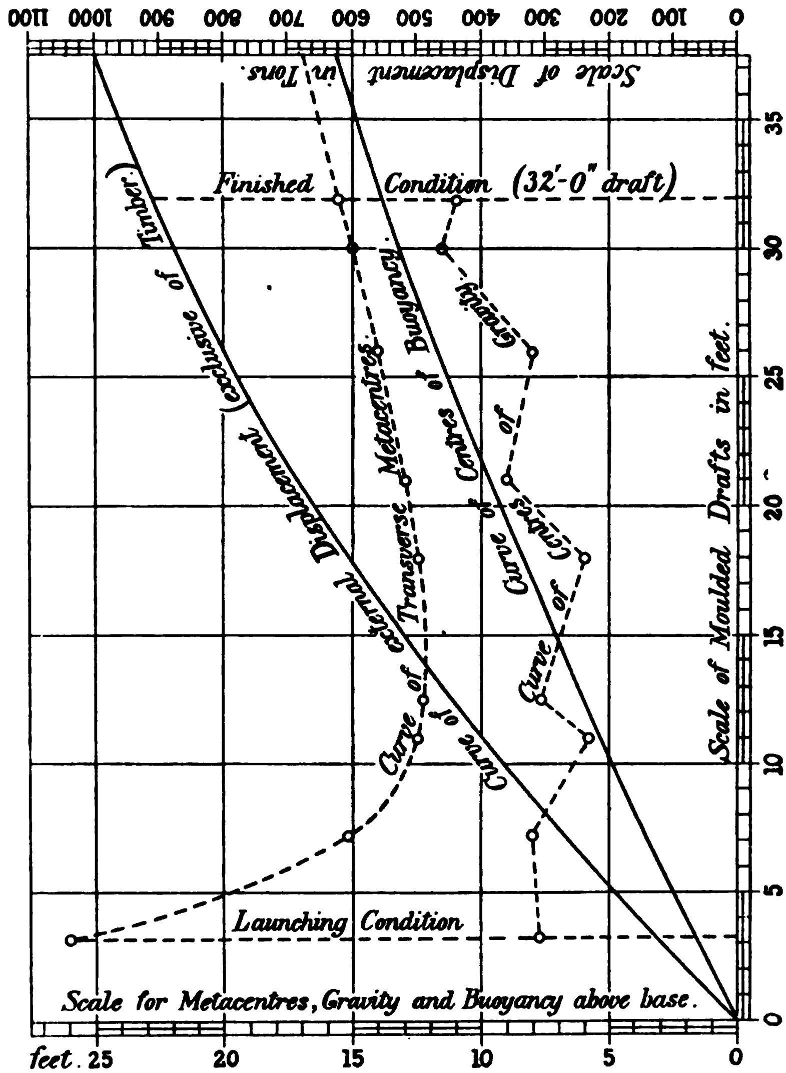


Fig. 892.

to the site, and final sinking in place, it was deemed desirable that a large amount of transverse stability when afloat should be maintained.

This was secured by the ballast stowed upon the floor, and between the floor girders, of compartments 1 to 6 inclusive. This

ballast consisted for the most part of burr concrete, composed of steel "burrs" or punchings grouted in with Portland cement mortar, and supplemented with ordinary concrete ballast. By this means a large metacentric height was maintained at all stages of construction, as shown in Fig. 392, which gives the curves of centres of buoyancy and gravity, metacentres, and displacement from the launching condition to that of final flotation and draught previous to sinking.

A summary of the weights at launching and at the finished draught of 32 feet is given in the following table:—

	Launching condition. Tons.	Final condition.
Net steelwork	219·87	383·66
Rivet and bolt heads	5·34	9·31
Paint	2·37	4·14
Timber	20·68	90·06
Bolts and plates for timbers ...	0·34	2·89
Burr concrete	33·00	412·50
Ordinary concrete	...	58·70
Sluices and mooring rings ...	...	2·99
Total weight	281·60	964·25
Draught forward	2′ 1½″	32′ 0″
Draught aft	4′ 6½″	32′ 0″

The conditions of stability at the final condition (32 feet draught) were as follows:—

Centre of gravity	11·12 feet above base
Transverse metacentre ...	15·60 „
Centre of buoyancy	13·80 „
Transverse G.M.	4·48 feet.

The burr concrete was about 2 feet thick over the floor, and the ordinary concrete placed above was about 9 inches thick.

The burr concrete,[1] 412·5 tons in weight, was composed of 370 tons of punchings and 42·5 tons of Portland cement and sand, the mixture weighing about 350 lbs. per cubic foot. Experiments made on the weight of concrete used in filling the caisson gave the following results:—

[1] For further remarks on Burr concrete, see p. 402.

1 Portland cement, 1½ sand, 5 limestone broken to pass through an 1½-inch ring, weighed 155 lbs. per cubic foot = about 14½ cubic feet per ton, the voids in the limestone being about 44 per cent. and those in the sand 30 per cent.

A concrete of 1 Portland cement, 2 sand, and 5 broken stone, gave nearly the same results, weighing 157 lbs. per cubic foot = say, 14¼ cubic feet per ton.

The caisson up to this point has been considered simply as a riveted steel structure, and further reference to its subsequent history and the ultimate use to which it was put might be regarded as outside the scope of these notes, if it were not that the subsequent operations explain certain peculiarities in the design. These, then, will be briefly alluded to.

As previously stated, the breakwater was founded upon a mound of quarried and deposited limestone rubble, in a depth of water varying from 45 to 65 feet below low water, and brought up to a level of 36 feet below low water, this being the depth at which it was considered the foundation courses of the superstructure could be laid with safety.

That portion of the surface of this mound upon which the caisson was intended to rest was carefully levelled by divers and brought to a fair surface by the deposit of fine material or small stuff, care being taken that no large stones projected above the finished level.

The caisson, having attained the draught before mentioned, was towed into position over the prepared site above described, and, the valves to the end chambers being opened, was sunk through the space, about 4 feet, intervening between the prepared surface of the mound and the under surface of the caisson at low water.

The possibilities of bad weather and heavy seas rendered this, perhaps, the most critical juncture in the entire scheme, and the succeeding operations about to be described were planned with the view of obtaining in the shortest possible time such a preponderance of dead weight over displacement as should reduce the risk of any shifting of the caisson out of its place by heavy seas to a minimum.

The programme described in the following table was therefore planned and carried out as closely as circumstances permitted, and with entire success, the concreting being carried on with the utmost possible despatch, while mooring chains were used as an additional safeguard against any shift of the caisson.

SUMMARY OF CONDITIONS AFTER SINKING.

No.	Conditions during the process of pumping and concreting.	Hull. Tons.	Water. Tons.	Concrete Tons.	Total weight. Tons.	Total buoyancy. Tons.	Excess weight. Tons.
1	Caisson sunk, and water in Nos. 1, 3, 4, and 6 compartments up to 40′	493	556	471	1520	1095	425
2	Water in compartments Nos. 1, 3, 4, and 6. Compartments Nos. 2 and 5 filled with concrete	493	556	1274	2323	1095	1228
3	Same as No. 2 for concrete. Water pumped out of compartments Nos. 1 and 6	493	278	1274	2045	1095	950
4	Water in compartments Nos. 3 and 4. Concrete in compartments Nos. 2, 5, 1, 6	493	278	1939	2710	1095	1615
5	Same as No. 4 for concrete. Water pumped out of compartments Nos. 3 and 4	493	—	1939	2432	1095	1337
6	Compartments Nos. 1 to 6 inclusive, filled with concrete	493	—	2604	3097	1095	2002
7	Same as No. 6, but with concrete seal in compartment No. 8, 5′ 8″ deep ...	493	629	2838	3960	1831	2129
8	Same as No. 7, but with water pumped out of compartment No. 8 ...	493	—	2838	3331	1831	1500
9	Compartments Nos. 1 to 6 inclusive, and No. 8 filled with concrete ...	493	—	4533	5026	1831	3195
10	Same as No. 9, but with concrete seal in compartment No. 7, 5′ 8″ deep ...	493	629	4767	5889	2567	3322
11	Same as No. 10, but with water pumped out of compartment No. 7 ...	493	—	4767	5260	2567	2693
12	Compartments Nos. 1 to 6 inclusive, and Nos. 8 and 7 filled with concrete	493	—	6462	6955	2567	4388
13	Same as No. 12, but with concrete seal in compartment No. 9, 5′ 8″ deep	493	629	6696	7818	3303	4515
14	Same as No. 13, but with water pumped out of compartment No. 9 ...	493	—	6696	7189	3303	3886
15	All compartments filled with concrete	493	—	8391	8884	3303	5581

An examination of the last column of the foregoing table, which gives the excess in tons of dead weight over buoyancy, will show the general increase in stability as the processes of pumping out and concreting went on, and the final result of the operations above described was the gradual building up of a monolithic structure, which, when first launched, had a total weight of about 281 tons, but in its completed state weighed nearly 9000 tons, and formed the first stage in the construction of a breakwater, of which the caisson itself formed a portion of the length, and was possessed of at least as much stability as any other section of the breakwater when finished.

Upon this caisson, completed in the manner described, were erected the Titan cranes,[1] by whose means the whole of the sloping blockwork was set. One Titan was first erected, as shown in Fig. 393, and, forming its own road as it advanced, was gradually worked off the caisson to make room for a second Titan of similar dimensions and power, working in the opposite direction to the first. By these two machines the whole of the remainder of the breakwater, including the heads, was constructed, and no further temporary staging of any kind was required, notwithstanding that the work was entirely cut off from the shore, with no communication, temporary or otherwise, except that of the floating craft transporting the concrete blocks from a blockmaking yard on the mainland.

The concrete blocks were laid in their sloping position by means of Fidler's patent block-tilting apparatus, designed by the author for this purpose.

It is not of course to be claimed that the system here adopted would be universally applicable, or would offer a sufficient guarantee against all sea risks in all situations. Varied exposures, and the local risks to be met, will obviously rule the decision as to the best method to be adopted; but any further consideration of this branch of the subject is outside the range of these notes.

Caissons for closing dock entrances.—The entrance to a graving or dry dock may be closed and rendered watertight by one or the other of two methods, viz. by gates or by some form of movable dam or caisson.

It is not within the scope of these notes to institute a complete comparison between these two methods of construction, or to enter

[1] For a description of these machines, the reader is referred to the article on "Titan Cranes" in the *Supplement to the Encyclopædia Britannica*, by Mr. Walter Pitt, M.I.C.E.

into all the arguments for or against either type on the grounds of relative economy, efficiency, or ready handling.

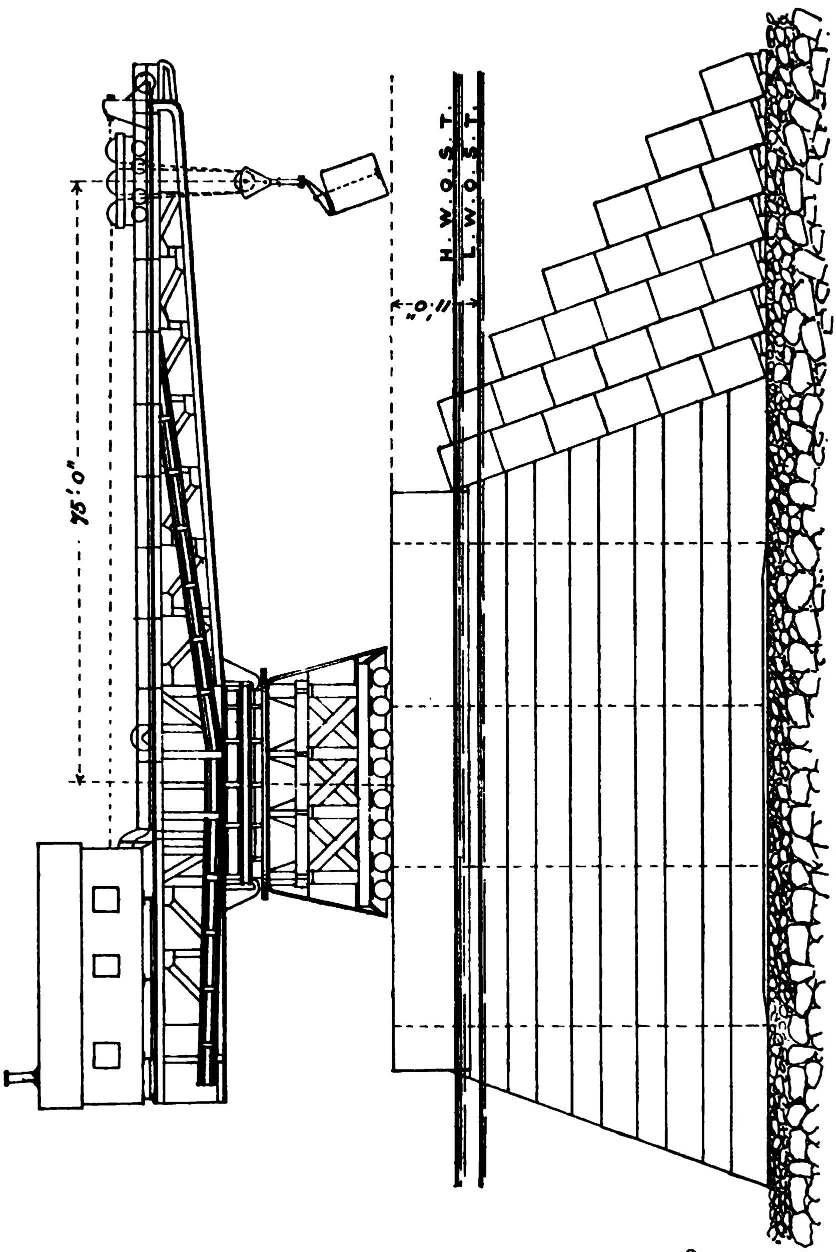


FIG. 393 (Scale 1 inch = 24 feet).

But for present purposes it may be pointed out that where the conditions of the site and the requirements of the dockyard or the commercial port are such as to demand a line of railway across the dock entrance capable of carrying with safety the heaviest rolling loads which have to be handled, such as boilers, heavy articles of machinery, guns, or the like, then the caisson possesses advantages over its rival, inasmuch as such a line of railway cannot conveniently be carried over a pair of dock gates of the ordinary design, but would demand a swing bridge of very considerable dimensions and cost, in addition to the gates themselves.

A bridge of this description would, moreover, frequently prove an obstruction to wharfing operations, and occupy valuable space.

The caisson, under such conditions, on the other hand, contains within itself the capacities of both gates and bridge combined, as the entire structure can easily be made of sufficient strength as a roadway to transport across the dock entrance the heaviest loads which can ever in practice be brought upon it, while at the same time fulfilling the functions of a movable dam and completely sealing the dock entrance against the maximum water pressures which can occur at the highest tidal levels.

Caissons for dock entrances may be divided broadly into two principal classes, viz. sliding and floating caissons, or, as the latter class is sometimes denominated, ship caissons, due probably to some fancied resemblance to ship forms of construction, especially when built on lines of curvature and outline approximating to those of ordinary vessels, although such outlines are sometimes adopted more from motives of appearance, or possibly of precedent, rather than from any practical advantages to be gained.

The sliding caisson derives its nomenclature from the methods adopted in the removal of the caisson from the dock entrance by sliding or hauling it into a recess in the dock walls prepared for its reception, and in such a recess the caisson is stowed away, and forms no barrier to the ordinary procedure of docking or undocking vessels.

The methods by which such sliding and hauling operations are performed will be further described, together with the precautions to be observed.

The floating caisson, on the contrary, is so designed in the mutual adjustment and balancing of its weight and displacement as, by a slight modification of the former, it can either be raised

out of, or sunk into, the grooves which are prepared for its reception in the sill and dock walls of the dock entrance, its final removal from the entrance being accomplished by warping or towing the caisson away to some convenient berth prepared for it.

It is in the necessity of providing such a berth that an objection is found in some cases to the employment of a floating caisson as compared with a sliding caisson. In the former case the space taken up by berthing the caisson in a basin or wet dock, or in a tidal stream, or alongside a wharf wall or jetty, may be of importance, although provision is sometimes made in the construction of the dock walls for a recess to receive the caisson, so as to be protected from the risk of grazing or colliding with passing vessels.

From this point of view the sliding caisson possesses the advantage, as above described, of being completely stowed out of the way, in its own recess, by means of the hauling apparatus provided for that purpose.

On the other hand, it is found that the cost of the sliding caisson is approximately twice that of the floating caisson, even if the cost of building the special recess is left out of the calculation, while the amount of machinery is greater, and the risks of a breakdown correspondingly increased, although it must not be overlooked that the floating caisson is exposed to certain risks of its own in the process of removal from, or reimplacement in its groove, which are not shared in quite the same degree by the slider.

Both types of caisson possess, or should possess, the capability, when properly designed, of removal from their position for the purpose of dry-docking for repairs and examination and painting or scraping.

For this purpose the sliding as well as the floating caisson must possess the requisite degree of stability as a floating body for all the usual operations of warping, towing, and docking; and practical experience has indicated what amount of stability such structures should possess when designed in the manner shown by the examples which follow, and which will be further discussed.

Whatever functions as a bridge which a caisson may be called upon to fulfil, either as means of transport for passenger traffic only, or as a roadway for mixed railway and road traffic of a heavy description, it must, as a dam at least, fulfil one important condition, viz. that of watertightness, and of keeping water out of the dry

dock at the highest tides with the least possible amount of leakage, and consequent demands upon the drainage pumps.

The problem of forming a watertight joint around three out of four of the sides of a superficial area amounting to as much as 5000 square feet, and having a lineal dimension of nearly 200 feet run, exposed to an hydraulic pressure which in some cases amounts to as much as 53 feet 6 inches of head, capable of being easily unsealed, and as easily sealed up again, is an interesting one of great practical importance, and were it approached from abstract mathematical grounds, or from the point of view of theoretical hydraulics, would afford scope for sufficient speculation.

Fortunately, the practical experience of many years has shown that the contact produced by the hydraulic pressure against the caisson, between two surfaces, the one of planed greenheart or other suitable hard timber, and the other of patent axed granite or other suitable hard masonry, is sufficient to produce a joint practically watertight over its entire superficies, and capable of being made and unmade with ease and celerity, while the amount of watertightness possessed by a well-designed and properly fitted caisson is greater than that possessed by the majority of the best constructed gates.

Upon the practical experience thus gained on this most important detail is based the design of the keel and stems of sliding and floating caissons, being the important preliminary in the design of the entire structure, these keels and stems having to supply the required reactions of the supports as against the hydraulic pressure of the full head of water acting against the entire exposed surface of the caisson at highest tides on one side, the other side being dry.

In Fig. 394 is given a detail of the arrangement of timber keels to a floating caisson, showing the contact of the timber with the masonry of the sill and the extent of the watertight seal. In this case the groove is 2 feet 3 inches in width, and a clearance of 4 inches is allowed, by which amount the caisson "fleets" over, when the direction of water pressure is reversed. In Fig. 395 is given the detail of the timber stems in the groove cut in the masonry of the sides of the dock entrance, this groove being a continuation of the groove cut in the sill, and of the same width, but 2 feet deep in place of 1 foot 3 inches, the 10-inch space shown in Fig. 395 being provided to enable the caisson to swing out of the groove, in combination with the additional clearance gained by

the batter of the sides of the entrance, when the caisson has floated up a few feet, the 10-inch space being an important factor in the operation.

These details show also the mode of connection of the timber

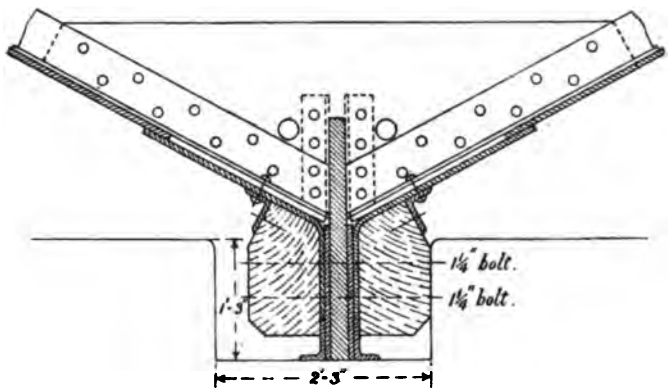


FIG. 394.
Scale ½ inch = 1 foot.

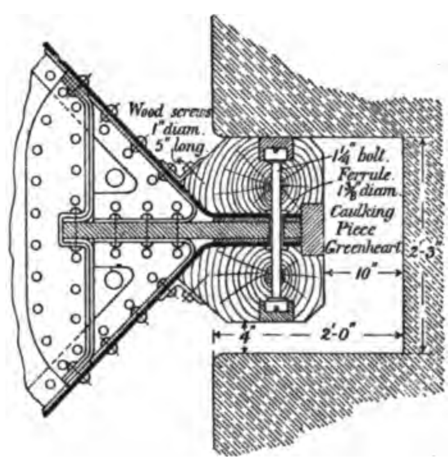


FIG. 395.
Scale ½ inch = 1 foot.

keels and stems with the framework and skin plating of the body of the caisson, to resist the severe shearing (or combination of shearing and bending) stresses which take place at this point. The method shown in Figs. 394, 395 has been successful in caissons having a total area of entrance up to 3500 or 3800 square feet.

Beyond this amount of area, and for greater water pressures, it has been deemed advisable to adopt the type of construction shown in Fig. 396, showing a stronger method of connection with the caisson body; but this type of keel demands a groove of greater width, as shown in the figure. It has been applied to dock entrances having a total area of opening of 5163 square feet.

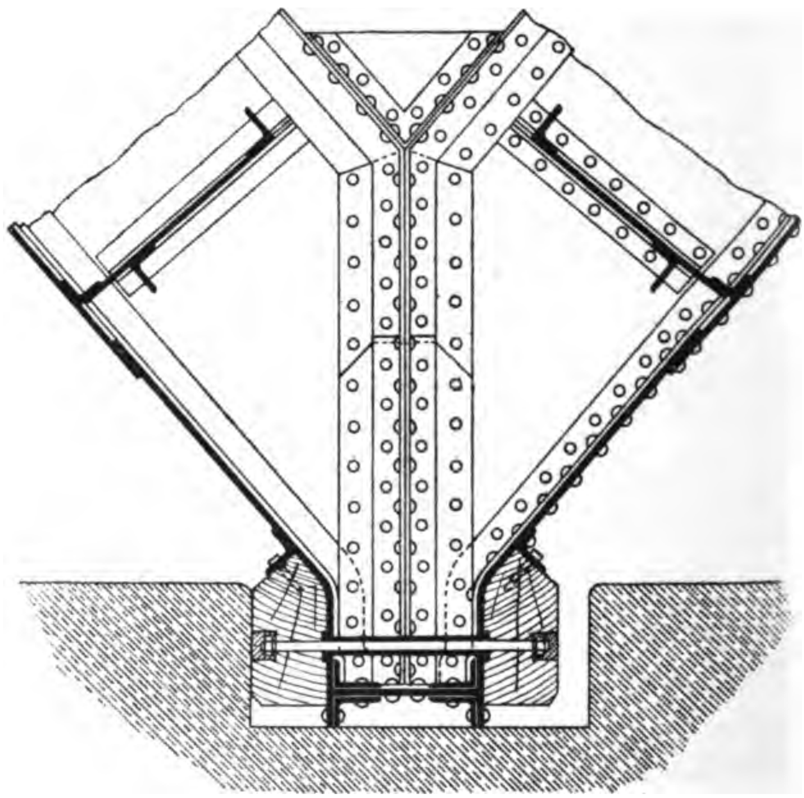


FIG. 396.
Scale ¼ inch = 1 foot.

The arrangement of timber keel for a sliding caisson is shown in Fig. 397, which gives the arrangement of the internal steelwork at this point, and shows the fine axed granite sliding way upon which the caisson moves, and the patent axed granite stop which forms the watertight seal with the timber keel.

In the above examples the net width of the surfaces in contact, after allowing for roundings or chamfers, is from 11 to 13 inches,

and this amount, under the severe pressures experienced, is found to give a satisfactory watertight joint.

The pressures to be sustained by the timber require a hard wood, and the exposure to the ravages of worm, a wood as little

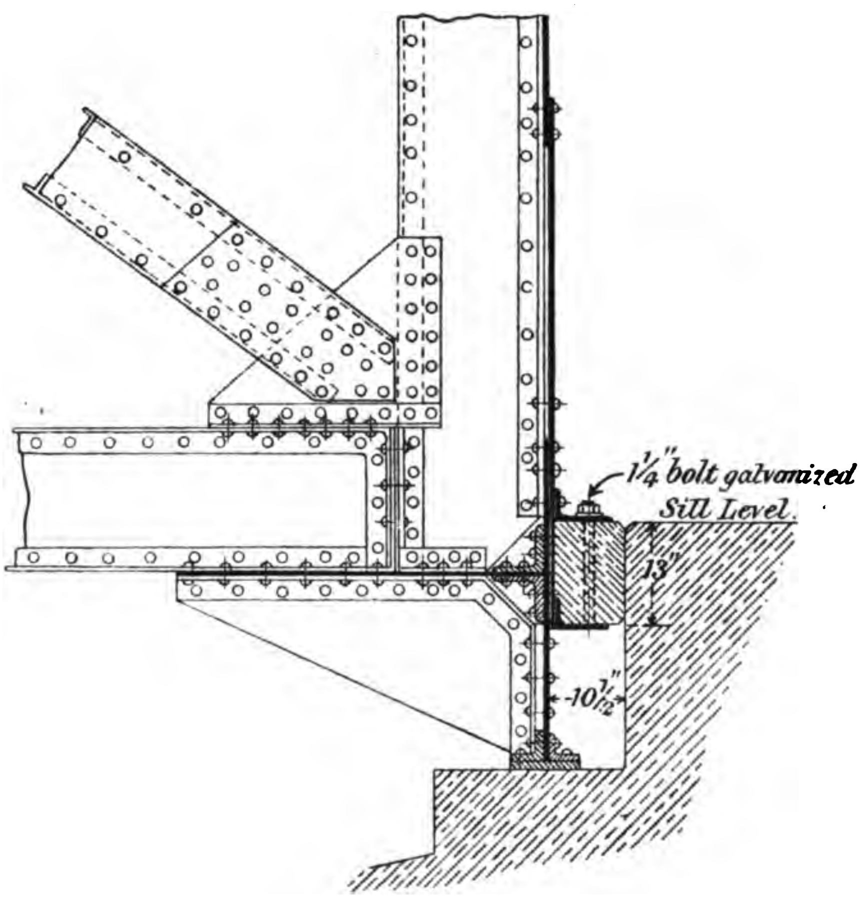


Fig. 397.
Scale ¼ inch = 1 foot.

susceptible as possible to these attacks, consequently the timber employed is very frequently greenheart, which also yields the required scantlings without difficulty. The timber used should be of the best quality, well seasoned, in long lengths, worked with square butts properly shifted with regard to joints in steelwork. The outer faces of the keels and stems should be planed to true

surfaces to meet the masonry faces with which they are in contact (in some cases the centre of the contact-face of the timber has been left a trifle proud).

The inner surfaces are carefully scribed over all angles or covers in the steelwork, and well bedded on canvas soaked in red lead paint so as to form a watertight joint.

The timbers are secured to the steelwork by galvanized bolts and coach screws, as shown in the details, the bolts, in the case of the floating caissons, passing through steel ferrules, and in the sliding caissons through the angle steels arranged to take the keel, as shown in Fig. 397. The heads and nuts of the bolts in the keels and stems of floating caissons are sunk into the woodwork, as shown, and covered with timber plugs set in marine glue pitch, the voids round heads and nuts being filled with pitch and sand cement.

The edges of the keels and stems are chamfered, as shown, to avoid splintering due to blows or rubbing against the masonry. The chamfers on the timber and the roundings on the masonry also facilitate the operations of removal and replacing of the caisson.

The details of the stems and keels have thus far been considered as a preliminary to the further stage of the design, viz. the question of the transverse strength of the entire caisson against the maximum water pressure on one side only.

The total amount of the reactions to be supplied by the pressure of the surfaces of the stems and keels is, of course, known, being equal to the total water pressure. But the precise value of the distribution of these reactions over each unit of the entire area of the surfaces in contact is not so easily determined. Assumptions as to distribution of stress may be made, for example, which would lead to the conclusion that the larger portion of the reaction was supplied by the sides of the dock entrance, leaving but little to come upon the lower sill; or, contrariwise, the sill might be supposed to be doing the bulk of the duty, leaving little for the sides, neither proposition being probably quite correct.

If the intensity of pressure at all points round the superficies of sides and bottom keels could be experimentally determined, the calculation of stresses would be simple; but, so far as the writer is aware, this has not yet been ascertained.

The solution is probably to be found in the elastic deformation of the entire structure, where the stiffness of both longitudinal

and vertical planes of girders is taken into account, and the distribution of stress is no doubt largely ruled by the rigidity of the principal planes of horizontal girders, which are the upper and lower decks of the air chambers in the types of caissons we are now considering.

These decks having to be watertight are plated, and thus constitute plate-web horizontal girders of great depth (in most cases) in proportion to their span, and having a corresponding rigidity, or small deflection under load.

But the positions of these decks is governed by the dimensions and position of the air chamber, and these latter are determined by considerations quite apart from those governing transverse strength, being ruled by the required buoyancies, and the influences of tidal levels, which will vary with every port.

The usual assumptions, though possibly devoid of mathematical refinements, of considering the entire structure as consisting of so many horizontal girders, each sustaining its proportion of water pressure according to the area exposed and the hydraulic depth, and assisted (where the internal arrangements permit) by other vertical girders crossing the former at right angles, are found to yield practically reliable and safe results, and at least indicate one mode of resistance which must be overcome before final rupture can take place.

By whatever mode of analysis the stresses in the entire structure may have been arrived at, the practical designer will consider the effects of corrosion upon immersed steelwork and the minimum scantlings which should be allowed.

The conditions of working of a modern dry dock frequently lead not unnaturally to the result that the caisson is the last floating structure to be cared for and docked. Repairs are postponed to a convenient period, which is sometimes long in coming, and the caisson has, as regards scraping and painting, to take its chance, for some years.

Such considerations are obviously opposed to any unnecessary refinements in the thinning down of scantlings, although, on the other hand, undue excess of weights must be avoided, as demanding correspondingly large buoyancies and increase in size of air chamber.

In the examples of caissons now before us the thicknesses of plates and bars range from $\frac{7}{8}$ inch or $\frac{3}{4}$ inch to $\frac{3}{8}$ inch, no thickness less than the latter being allowed; and, broadly, it may be

stated that from $\frac{1}{8}$ inch to $\frac{1}{16}$ inch more metal is allowed than merely theoretical requirements would determine.

There may be, and probably are, examples of this class of work in private establishments, where such a view of the maintenance of the life of a caisson or gate is not taken, and where considerations of first cost have influenced the selection of scantlings on a less liberal scale.

With these preliminary remarks on the general subject, we may now consider in detail certain examples of both classes of caissons, floating and sliding, the former being taken first in order, being somewhat simpler in construction than its rival.

The type of caisson shown in midship section in Fig. 398 consists, as regards its internal arrangements, of five principal subdivisions, the lowermost of which, termed the bilge, is open to the water, by means of a series of flood openings on one side only through the skin, placed below the lower deck of the air chamber.

The bilge contains in its lowermost position, just above the keel, the ballast necessary for the stability of the caisson when floating, and is so arranged as to be capable of being drained dry into the dock when the latter has been pumped out, suitable outlets with plugs being provided, the key for opening which can only be withdrawn when the outlet is closed.

Above the bilge the next subdivision to be observed is the air chamber. This chamber is of capacity sufficient, when combined with the buoyancies of all immersed materials, to float the caisson, with all weights and ballast on board, at the level of the upper deck of the air chamber.

The level of this upper deck is consequently determined by the lowest level of tide at which it is required that the caisson shall be capable of being removed from the groove, and by the amount of vertical lift necessary to enable the stems of the caisson to clear the corners of the groove. The amount of lift required will depend upon the degree of batter in the side walls of the dock entrance, together with the clearance left between caisson stems and back of groove, referred to and shown in Fig. 395.

If, for example, a lift of, say, 5 feet is necessary before the caisson can be safely manipulated out of its grooves, then the level of the upper deck of air chamber, being the flotation plane, must be 5 feet below the tidal level at which it is determined that the caisson shall be worked, and this tidal level will usually have

some relation to the depth at which the sill of the dock has been laid.

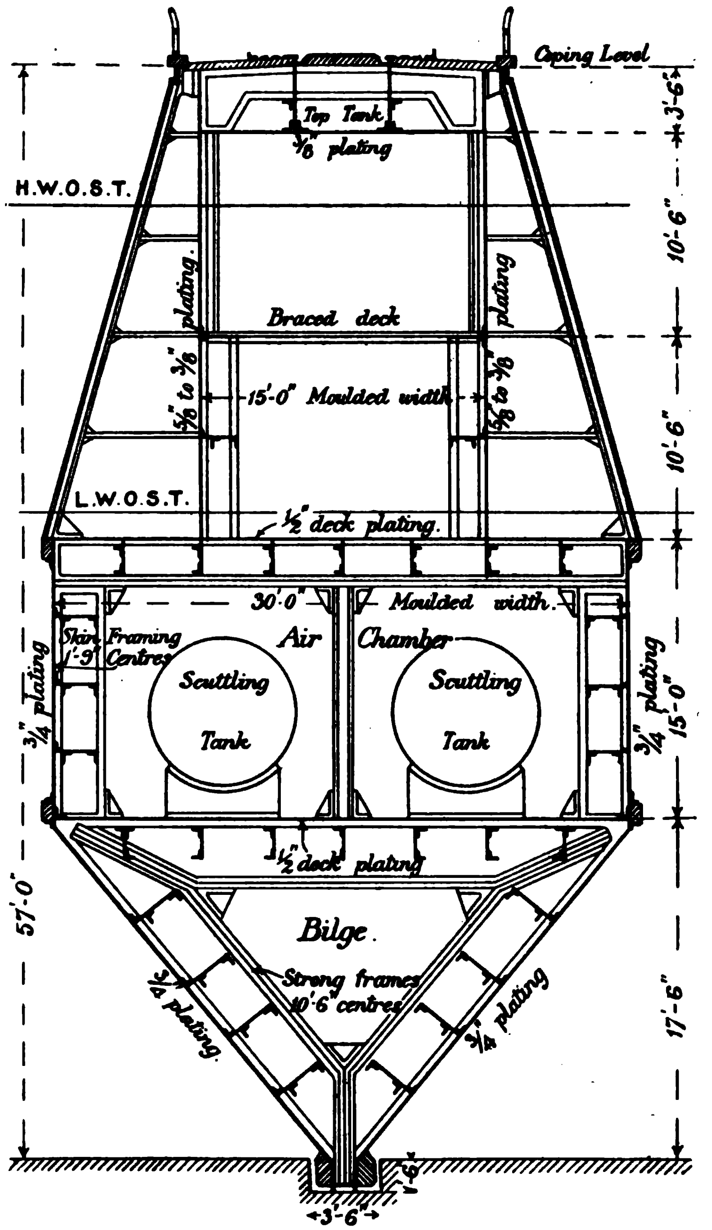


FIG. 398.
Scale 1 inch = 10 feet.

The position of the air chamber having thus been determined in the relation of its upper deck to the total height of the structure from sill to coping, the volume of the chamber is then determined in accordance with the buoyancy required, and the level of its lower deck will follow.

But the conditions of practical working demand a certain stability of the caisson when floating, and practical experience has shown that a pendulum of from 1 foot 6 inches to 2 feet 6 inches, in accordance with the size and total height of the structure, is desirable; that is to say, that the centre of combined buoyancies (C.B.) shall be from 1 foot 6 inches to 2 feet 6 inches above the centre of combined weights, or the centre of gravity (C.G.) of the caisson when ballasted. This pendulum will be reduced if the caisson is worked at any time with the top tanks full. Further reference to these tanks will be made.

It is thus seen that the capacity and position of the air chamber must be such as to meet all the conditions above described, as the amount of ballast required for the stipulated stability is frequently considerable, especially where, in consequence of the relative position of tidal levels and the conditions as to floating out, the air chamber is comparatively low down in the structure.

The air chamber will be frequently found to contain scuttling tanks, which, being flooded, reduce the buoyancy of the chamber, and are usually brought into action when rapid sinking is desired, or to counteract excessive buoyancies under certain conditions of working. Such tanks must, however, be emptied before the caisson can be restored to its normal working condition, and this implies pumps, which may be worked by compressed air, electricity, or other power, commonly supplemented by hand gear, to be applied in case of a breakdown. The weights of all such internal machinery must, of course, be taken into the account when estimating the combined centre of gravity (C.G.) of the structure.

In the example before us, the scuttling tanks in the air chamber consist of two steel cylindrical tanks or boilers, seated on stools attached to the lower deck of the air chamber, stayed to prevent any motion consequent upon heeling over of the caisson, and so arranged as to be blown out and emptied of their contents by compressed air, admitted by suitably arranged pipes, and connected at the upper deck level by means of flexible high pressure hoses to the compressed air mains adjacent on the dock side.

Communication between the tanks and the sea is maintained at all times by pipes and valves worked from the upper deck.

The blowing-out apparatus is associated with hand pumps, the latter being used in case of breakdown or failure of the compressed air supply.

The space above the upper deck of the air chamber is subdivided into three compartments, separated by watertight bulkheads; the central compartment communicates with the bilge by two watertight trunks passing up through the air chamber, so that the water in the central division will rise or fall with the level of the water outside, which is in communication with the bilge by means of the flood-holes before mentioned.

The two end compartments are not in communication with the bilge, but are connected by trunks and valves controlled from the upper deck, with the water on the side opposite to the flood-holes. These compartments can therefore be flooded to any extent desired, and control is thereby obtained over the longitudinal trimming of the caisson, should such be necessary. Water admitted into the end chambers at a high tide can also be retained and locked in, an essential element under certain conditions, or at such times as the caisson is turned end for end in the grooves.

The uppermost deck of the caisson is designed to accommodate a mixed traffic of passengers and horses or carts, together with a single line of rails calculated for the heaviest rolling load which can occur in practice.

This load has sometimes been taken as high as 72 tons on four wheels, having a wheel base of 6 feet 6 inches, and the whole of the rail-bearers and cross-girders have been calculated on this basis. The remainder of the deck is laid with oak planking, with steel wheel tracks and guards for ordinary wheeled traffic, horse treads being applied between the rails, while the various trunks and manholes, openings for valve spindles, etc., are covered with movable or hinged flaps.

The groove in the masonry of the dock sides and sill for a floating caisson is usually about 4 inches wider than the outside width of the keel or stems, and the caisson "fleets" over from side to side of the groove by this amount, as the water pressure is on one side or the other. See Figs. 394, 395. To provide for this movement an automatic switch is usually provided, whereby the rails on caisson deck are kept in correct adjustment with those on the dock side.

In the case of flaps over valve spindles, an automatic arrangement is frequently adopted whereby any condition of the valves below which implies danger to the stability of the caisson by reason of a temporary excess of buoyancy (the danger to be the most carefully guarded against) is indicated by the flap standing up, and its inability to be closed until the valves have been adjusted to a "safe" position, while such a position of the flap acts as a danger signal to the caisson master.

Handrailing is provided on both sides of the upper deck, and is usually arranged to be turned down on the roadway when required, so as to be out of the way during warping or docking.

Immediately below the uppermost deck, but placed as high as possible, so as not to be overlapped by the highest recorded tides, is constructed the top tank before referred to. The function of this tank is to contain a sufficient amount of water, either salt or fresh, as may be convenient, to overcome and destroy the buoyancy of all immersed material above the level of the upper deck of the air chamber, and it is by the filling or emptying of this tank that the caisson is caused to sink into, or rise out of, the grooves.

The tank is usually filled by hose from water mains on the dock side, and is emptied by valves controlled from the upper deck.

When a floating caisson has been removed from its groove, it is sometimes found convenient, as a saving of time, to fill the upper tank prior to warping back again. This may be done, providing the stability of the caisson permits, by closing the valves to end chambers, thereby providing the additional buoyancy required, but adding to the draught of water.

These valves being opened, when the caisson is in its groove, the buoyancy of the end chamber is destroyed and the caisson sinks.

In the type of caisson shown in Fig. 398 the same result can be obtained by the admission of water into the cylindrical tanks in the air chamber, the valves to end chambers being closed. In this case, before the caisson can rise again the contents of the cylindrical tanks must be either blown out or pumped up.

It has been previously hinted that one of the most serious dangers to which a caisson can be exposed is a temporary excess of buoyancy, whereby an upward lifting tendency is developed.

This has led to serious accidents in certain cases, resulting in

the forcing-in of the caisson and the immediate flooding of the dry dock, and consequent risk of damage to vessels on the blocks.

Caissons unsymmetrical about their centre line, with a flat side to the dry dock, and the whole of their displacement on the tidal side, are perhaps most exposed to this risk; but in all classes of caissons the point is deserving of the most careful attention, and calculations for buoyancy should always include the worst possible conditions as regards the proportion of weight to buoyancy, when, for example, the bilge has been drained into a dry dock, and the readmission of water into the dock produces a set of conditions temporarily out of the normal.

To meet such contingencies, and to provide for the accidental emptying of tanks through leaky valves or the like, holding-down straps, bolted to a sufficient mass of masonry at the upper part of the side grooves, are frequently adopted, so arranged as to hold down the caisson, and provide an anchorage sufficient to supply the place of any temporary loss of weight resulting in an excess of buoyancy.

It will generally be found that before a serious excess of buoyancy can arise, a difference of head of water, as between the two sides of the caisson, affords an additional safeguard against uplifting by reason of friction between the stems and keels of the caisson, and the faces of the groove.

A coefficient of friction, even when derived from careful experiments on faces prepared to represent the normal working conditions, is, however, likely to become uncertain in its amount, if any alteration in the condition of the two faces in contact should take place in course of time under the actual working condition of immersion in sea water, and instances are not wanting where the frictional resistance has been entirely overcome by the uplifting tendency of the caisson.

It is prudent, therefore, in the designing of these structures to ignore the additional safeguard due to frictional resistance, and to rely upon the ascertained proportion between weight and buoyancy, supplemented by the holding-down power of the anchorages above alluded to.

The coefficient of friction between planed greenheart and patent axed granite has been ascertained by experiments carried out on a full size scale to be about 0·27 for dry timber upon dry masonry, and about 0·39 for wet timber upon wet masonry, these values representing the coefficients in each case for surfaces

prepared to represent those actually occurring in practice when new, but not exposed to long-continued immersion.

The calculations necessary for the complete design of both floating and sliding caissons, apart from those required in the determination of the transverse strength against water pressure, are apt to be lengthy and tedious, as there is no royal road to the accurate determination of the centres of gravity and buoyancy, and average dimensions derived from existing examples should be used with caution, and for preliminary investigations only. The whole of the items of riveted steelwork and other materials should be carefully calculated for weight, and the lever arms of each separate item above some convenient datum plane ascertained, and worked out to obtain the sum total of moments about the given plane. From these the centre of gravity of the whole will be obtainable, and a similar course is necessary for all buoyancies, including immersed material and ballast.

The percentage of rivet heads and points is an important item, and for the heavy class of work customary in caissons with close-pitched watertight riveting should be taken as not less than 4½ per cent., while all immersed timber should be calculated at its saturated weight, if the gross bulk of the timber be reckoned for buoyancy. Recent experiments on the weight of greenheart, American elm, and Dantzic oak, saturated and dry, have yielded the following results:—

TABLE No. 36.

WEIGHTS OF TIMBER, WET AND DRY.

GREENHEART TIMBER.

	lbs.	ozs.
Weight per cubic foot when dry	71	12
,, ,, ,, after 7 days in water ...	73	5
,, ,, ,, after 1 month in water ...	74	11
,, ,, ,, after 2 months in water ...	75	5

AMERICAN ELM

Weight per cubic foot when very dry	57	5
,, ,, ,, after 7 days in water ...	60	14
,, ,, ,, after 1 month in water ...	63	12
,, ,, ,, after 2 months in water ...	65	10

DANTZIC OAK.

	lbs.	oz.
Weight per cubic foot when very dry	39	0
" " " after 10 days' immersion in water	53	0

Subsidiary items, such as paint, protective composition, tar, asphalte, cement, and the like, must also be allowed for, as well as their displacements.

A table is given at the end of this section, on p. 403, which gives the actual weights of recently constructed floating caissons of the type shown in Fig. 398, and of other types differing only in the arrangement of internal bracing and in the external contours. The examples given are of varying depths over the sills of the docks, and the weights given are the totals for riveted steelwork and other materials and machinery, but are exclusive of ballast of every kind, the amount of which in the examples selected ranged in round numbers between 150 and 300 tons, according to the conditions imposed by tidal levels and the position of the air chamber.

In order to obtain the measure of stability and length of pendulum previously alluded to, it will be found necessary to stow all ballast in the bilge at as low a level as possible.

The conditions of stowage, however, are not favourable to a great density per cubic foot, if pig-iron is used, owing to the breaking up of the space by reason of the numerous frames, diaphragms, etc., found necessary at this point, unless special castings to fit the spaces are employed, and it is convenient to use a material better adapted to give the greatest possible density per cubic foot when in position. The material known as burr concrete fulfils these conditions, the principal objection to its use being the difficulty of removal when once set hard. This is met by not using more than suffices to fill up awkward spaces in the constructional work, and to provide a level platform upon which the ordinary pig ballast can be stowed without excessive loss of space in the interstices.

The following table gives the results of experiments to ascertain the stowage value of pig-iron and other ballast under various methods of treatment:—

TABLE No. 37.

Item.	Description of ballast.	Weight per cubic foot in lbs.	Cubic feet per ton.	Percentage of interstices.
1	Rough pig-iron, about 3′ 6″ in length, and of ordinary section about $4\frac{1}{2}' \times 4''$, laid in rows in alternate directions, and stowed as close as possible	284·05	7·88	37
2	The same pigs broken into short lengths of 12″ and under, laid in rows in alternate directions and stowed as close as possible	287·22	7·79	36
3	The same as No. 2, the interstices being filled up with steel burrs	333·88	6·71	27
4	Steel punchings (burrs) alone	303·0	7·39	38
5	Steel burrs grouted with Portland cement mortar, 1 of cement to 1 of sand, and rammed (burr concrete)	350·0	6·40	Practically solid

Burr concrete ballast is composed of the "burrs" or punchings from steel plates or bars,[1] and when grouted together with Portland cement mortar, in the proportion of one of Portland cement to one of sand, becomes a dense conglomerate, which with care and sufficient ramming can be made to weigh 350 lbs. per cubic foot, and is capable of being packed into spaces too confined to admit of very close stowage of ordinary pig-iron ballast.

Although in itself this material forms a protective coating to steel, it is usual to prepare the faces of steelwork to receive the ballast by a coating of asphalte cement laid on all horizontal or vertical surfaces about $\frac{3}{8}$ inch thick. The composition of this cement is further described in Chapter VII.

[1] See Chapter III. p. 109.

TABLE No. 38.

THE WEIGHTS OF MATERIAL AND MACHINERY IN FLOATING CAISSONS.

Number of example.	Width of dock entrance at coping level.	Depth of dock entrance, coping level to sill level.	Total area of dock opening to masonry outline.	Weight of caisson per square foot of area of opening, exclusive of machinery and ballast. Tons.	Total weight of caisson per square foot of area of opening, including machinery, but exclusive of ballast. Tons.
1	95′ 4″	57′ 0″	5163	0·143	0·149
2	95′ 0″	55′ 0″	4970	0·145	0·150
3	95′ 0″	47′ 0″	4278	0·131	0·133
4	94′ 0″	42′ 11″	3796	...	0·116
5	94′ 0″	39′ 0″	3500	...	0·111

Sliding Caissons.—Much of that which has already been remarked previously in connection with floating caissons may be taken to apply also to sliding caissons, with respect to such items as the calculations for centres of gravity and buoyancy, the mode of ballasting, the percentage of weight for rivet heads, the transverse strength, the necessity of carefully providing against an excess of buoyancy under all possible conditions, and the method of making a watertight joint.

In respect of the last item, the details of the timber keels and facings of the masonry will be similar in kind (see Fig. 397), while the arrangements of the roadway over the upper deck, the strength of rail girders and bearers for the given rolling load, and the switch arrangements for providing an automatic continuity in the rails when the caisson fleets over (about 3 inches in the case of sliders), will be found generally similar in both types.

As regards other details of the upper deck, some variation in practice is found to occur, due to the various methods employed in providing for the hauling in of the caisson underneath the camber deck.

The camber, or recess in the dock side, into which the caisson is drawn, is of considerable dimensions, and occupies a space which must be decked over so as not to diminish valuable wharfing space, and to provide for the continuity of the road or railway across the upper deck of the caisson.

This camber deck is sometimes made a fixed structure, sometimes partly fixed and partly capable of being lifted, and latterly,

in recent examples, has been made to lift for its whole length, being hinged at the inner end, and lifted to the required amount at a convenient point near its outer end by hydraulic rams or other mechanical appliance, in accordance with the nature of the power used.

In the first case, where the camber deck is fixed, the upper deck of the caisson, when designed for foot passenger traffic only, is sometimes kept low enough to pass under the camber deck when hauled in. Where railway and wheeled traffic have to be provided for, this method is not applicable, and the caisson deck is made to fall and rise by means of special apparatus, deriving its power usually from the hauling engines, and acting automatically.

In these cases the sliding ways or rollers upon which the caisson travels are kept level, and the caisson maintains a level course throughout its travel.

Such means of raising or lowering the upper deck of the caisson are found to necessitate a considerable amount of top weight, if the rolling loads to be carried are considerable, and the scantlings of the upper deck correspondingly heavy.

Another method is therefore sometimes adopted, whereby the necessary clearance between the under surface of the camber deck and the upper surface of the caisson deck, when in camber, is obtained by lifting the camber deck either in part or entirely (as previously described), at the same time causing the caisson to travel on a descending plane into the recess.

When the travel of the caisson is completed, and the dock entrance completely opened, the camber deck is then lowered on to its bearings, and the wharfage space is left entirely unencumbered.

On the return journey, the camber deck being again lifted, the caisson rises up the incline to its normal position across the dock entrance, the camber deck is let down, and the line of rails and roadway are continuous and horizontal for the entire combined length of the caisson and its camber.

It may be said that the provision of such an incline throws greater duty upon the hauling engine, and theoretically this is true; but numerous observations of the actual hauling power required show that the additional work done is but a small proportion of the whole, and will influence the design of the hauling gear in only a very minor degree.

It is, of course, to be understood that in the type of sliding

caisson now under consideration, the greater portion of the weight of the caisson on the sliding ways or rollers is counterbalanced by the buoyancy of the air chamber and that of the immersed material.

The actual surplus weight on the ways or rollers can therefore be adjusted by ballasting so as to give the desired amount, which should be the least, compatible with general steadiness in working, and stability as against overturning moments produced by currents on a falling or rising tide, or temporary small difference of water level.

The frictional resistances caused by this surplus weight on the ways, together with the power required to move the entire mass at the given speed and to overcome all the resistances of the gearing employed, are the measure of the hauling power to be employed.

The functions of the air chamber, then, in sliding caissons are the same as that for floating caissons, and for docking purposes the slider is designed to float out of its position at the level of the upper deck of the air chamber (the hauling gear having been disconnected), and the same measure of stability or length of pendulum between centres of buoyancy and gravity is desirable.

As in the floating caisson so also in the sliding caisson, it is sometimes considered advisable to provide in the air chamber tanks for use in emergencies, by whose means the weight of the caisson can be increased by letting water in without flooding the entire air chamber. This also implies the provision of pumps worked by convenient power, such as steam, compressed air, or electricity, to get rid of the surplus water and restore the caisson to its normal working condition.

Such arrangements are more usual in cases where a large range of spring tides occurs, accompanied by exceptionally high tides above the normal high water spring tide level.

One of the most important features of the sliding caisson is the mechanism by which the caisson is hauled into and out of its camber or recess.

This is most usually accomplished by a pair of endless chains working over sprocket wheels (or drums, in accordance with the type of chain adopted) and actuated by powerful engines placed at the further end of the caisson camber, in a chamber below coping level specially prepared for its reception.

The motive power of such engines is usually either hydraulic, compressed air, or electric, supplied to the engines from some

central station equipped with hydraulic pumps and accumulators, compressed air pumps and receivers, or electrical generating apparatus.

The detailed design of such engines is somewhat outside the scope of these notes, and need not be further alluded to, except to state that a liberal margin of power is always judicious, in order to deal with contingencies, such as the increase of resistance on the sliding ways due to deposited mud, or temporary obstruction, or the jamming of the caisson stems and keels against their faces by the tidal outflow or inflow of water through the dock entrance.

With respect to the latter contingency, it must be remarked that ample culvert area should always be provided in the masonry construction of the dock as an essential feature in the safe and proper working of the caisson. A closed dock in direct communication with tidal waters will always demand special attention to these details, especially if the tidal range be considerable, as if insufficient culvert area is provided, the water can neither flow into the dock on a rising tide, nor flow out on a falling tide with sufficient velocity to maintain the water level on both sides of the caisson.

The resulting small head of water on one side or the other will be sufficient, if it exceeds certain limits, to prevent the opening of the caisson altogether, or, in a lesser degree, to increase the power required for, and add a certain amount of risk to, the operation.

In practice it is found that while a caisson may be workable with one to two inches difference of water level, a difference of as much as four inches might render it unsafe to attempt to open the caisson. Hence the necessity of a sufficiency of area in the culverts, and, it may be added, of a correct hydraulic coefficient in the calculations for flow and capacity. To meet any doubt in these respects, sluices are sometimes arranged in the body of the caissons themselves to supplement the culverts in the dock sides. Difficulties of the kind here described are, however, very frequently reduced to a minimum by limiting the operation of opening and closing the dock entrance within the period of slack water at the top of the tide, or, where the depth of sill admits, at low water. Sluices have also been provided in caissons for the purpose of the removal of mud by sluicing, but such methods are not usually now considered the most effective methods of removal of deposit, which is more generally dealt with by dredging operations of the usual character.

It is desirable that all portions of the hauling machinery should be above high water as far as is practicable, for observation and access for repairs.

This requirement implies that the point of application of the hauling force to the caisson body is at a high level above the points of resistance at the sliding ways, and the pull on the chains has a correspondingly large tilting moment, due to its leverage. This tendency to tilt or "kick" must be met either by appropriate ballasting of the caisson, or by a reduction in the frictional resistance of the sliding ways, or both combined.

This requirement leads us to some further consideration of the details of the sliding ways or roller paths upon which caissons of this type travel, and it will be found that a considerable diversity of practice exists.

On the one hand, numerous caissons have been successfully handled for years, which simply slide upon the prepared surfaces of granite ways or cast-iron plates, the keels of the caisson bearing upon the ways being simply sledge runners, while, on the other hand, it has been considered necessary to mount the caisson upon a continuous series of fixed rollers under water, and in one recent modern example so many as forty-four rollers have been thus employed.

In other cases the caisson has been mounted upon wheels, travelling upon rails of a suitable section.

The sledge-runner principle is of an exceedingly simple type, not liable to give much trouble, but involving a considerable amount of friction to be overcome, while the wear on the steel plates has been found to be considerable in the course of years, especially at the "toe" or forward end, where the pressure is increased by reason of the tilting moment of the hauling chains.

On the other hand, a multiplicity of rollers with their axles and bearings under water, although reducing frictional resistance, is open to the objection that difficulty may be caused if the bearings should rust up after prolonged immersion.

In certain recent examples, including some of the largest sliding caissons in existence, a combination of the two types of the sledge runner and the roller has been designed by the author and introduced with success. This arrangement is shown in Fig. 399, which is a transverse section of a sliding caisson represented by example No. 10 in the table on p. 427.

The caisson is furnished with a pair of keels, consisting of a

flat bar, 9″ × 1″, countersunk riveted to steel angles on the skin of the caisson, as shown in Fig. 397. These keels slide upon prepared granite ways, as shown, laid at an inclination of one in forty. But, in addition to the keels, two cast-steel rollers are provided on the centre line of the caisson, one near each end, running upon a cast-steel path laid on the centre line of the caisson ways for the entire length of its travel, both across the entrance and in the caisson camber. The relation between the level of the prepared granite ways and the upper surface of the cast-steel pathway is very carefully adjusted and maintained, so that the three plane surfaces are strictly parallel at the same inclination, and at a certain fixed distance apart vertically (see Fig. 399).

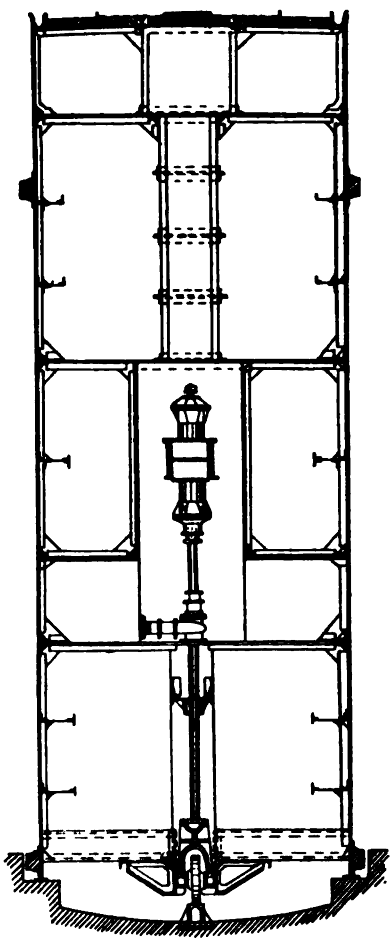


FIG. 399.
Scale 1 inch = 10 feet.

The rollers are operated upon by a pair of hydraulic rams, placed one near each end of the caisson. These rams are worked by pumps and manual labour at that end of the caisson farthest from the hauling engines, while valves are attached to the pumps so as to enable the pressure to be placed upon the rollers alternately, or, if necessary, on both at once. Thus, if the caisson is being hauled into the camber, the cylinder nearest the hauling engine (the forward cylinder for the time being) is subjected to hydraulic pressure, and the caisson keels are lifted by a small amount (about ½ inch to ¾ inch) off the granite ways, the whole weight being sustained by the roller and central pathway.

By the whole weight is, of course, understood only the surplus of weight over the buoyancy of the air-chamber and the immersed materials, an amount which will vary with the state of the tide and the weight of water in the upper tanks.

By these means the sliding friction of the keels is converted into the rolling friction of the roller, and the result is a large diminution of pull in the hauling chains, a corresponding reduction in the amount of the tilting moment, and a saving of wear and tear upon engines and gearing.

As the after roller has no weight upon it, or only as much as may be deemed desirable without lifting the caisson, the keels at the after end remain in contact with the granite ways, and the caisson being supported on three points is stable, roughly resembling the condition of a wheelbarrow when being pulled along with its legs upon the ground wheel foremost, the friction of the keels at the after end being, however, reduced by that proportion of the weight which is taken by the after roller.

On the return journey these conditions are reversed, end for end, the pressure being released from the roller nearest the hauling engine (the after roller for the time being) is transferred to the now forward roller, and the same results ensue, while the alteration in the conditions is obtained simply by the turning of a valve, together with any additional pumping up of the rams required by leakage, etc.

At the close of either journey both cylinders are relieved from pressure, and the caisson then settles down upon its keels and upon the granite ways, the rollers and central path being relieved of their duty. On this principle there are only two rollers to be kept in order, and the examination and repair of them is provided for by the arrangement of the hydraulic cylinders. By an adjustment of the valves above mentioned, the pressure can be transferred to the under side of the rams, and the rollers lifted off their pathway to the level of the lowermost deck of the caisson, where access can be had to them by divers, the stroke of the rams being made sufficiently long for this purpose.

Arrangements are also made by which the rollers can, if required, be detached, hoisted to the uppermost deck, and landed, without docking the caisson.

In Figs. 400 and 401, sections through various portions of the caisson are given, and Fig. 402 is an end elevation. Fig. 401 shows the general system of internal cross-bracing, forming a series of

vertical girders, square to the planes of the solid plate girders which form the watertight upper and lower decks of the air chamber.

The central cast-steel roller path shown in Fig. 399 is further shown in detail in Figs. 403 and 404, Fig. 403 being a cross-section of the casting, showing its attachment to the bed stones, the

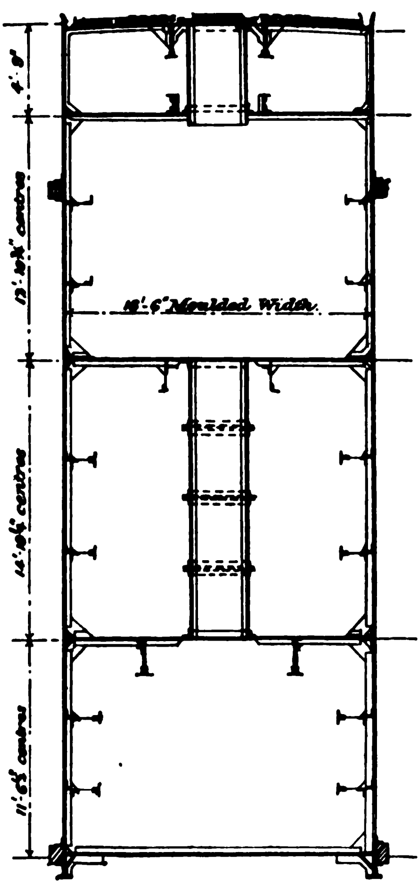


Fig. 400.
Scale 1 inch = 10 feet.

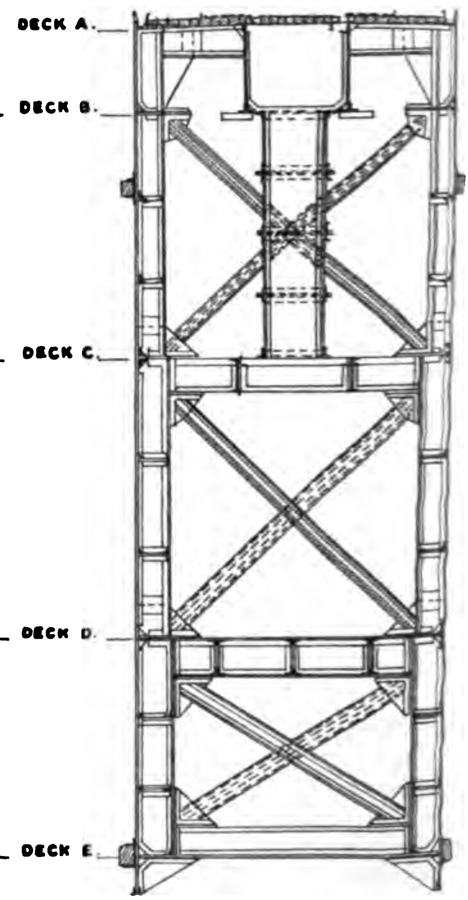


Fig. 401.
Scale 1 inch = 10 feet.

holding-down bolts being run with lead. Fig. 404 shows a plan and side elevation of one length of the roller path.

These castings are in lengths of 9 feet 7 inches, a dimension which experience in the foundry proved to be satisfactory for this section and material.

The lengths of roller path are machined on the upper and bearing surfaces, while the ends are machined by a special tool to the birdsmouth joint shown in the figure, this form of joint being adopted in order to mitigate the shock of the concentrated rolling load in passing over the joint from one length of path to the next.

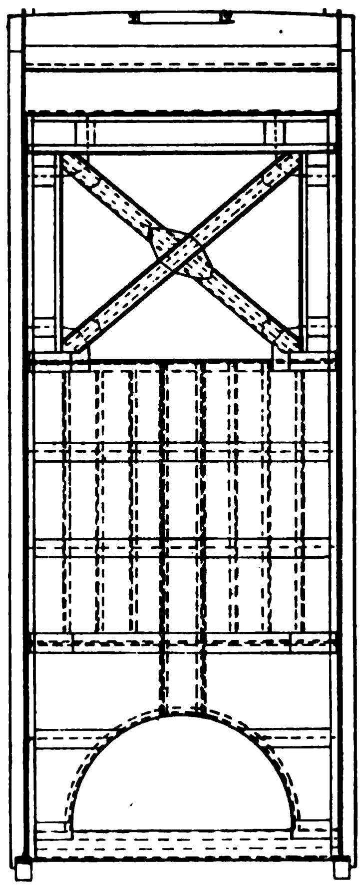


FIG. 402.
Scale 1 inch = 10 feet.

The quality of cast steel from which these castings are made is well shown by the results of numerous tests in the table of tensile and bending tests on cast steel bars, p. 69. The test bars were cast with the lengths of roller path, and cut off after annealing for testing purposes.

The bolt-holes at joints are bored, and bolts turned, with the object of obtaining as rigid a connection between the lengths as possible.

The width of tread of the cast-steel roller travelling over the path is 6 inches on the flat, and the upper surface of the path

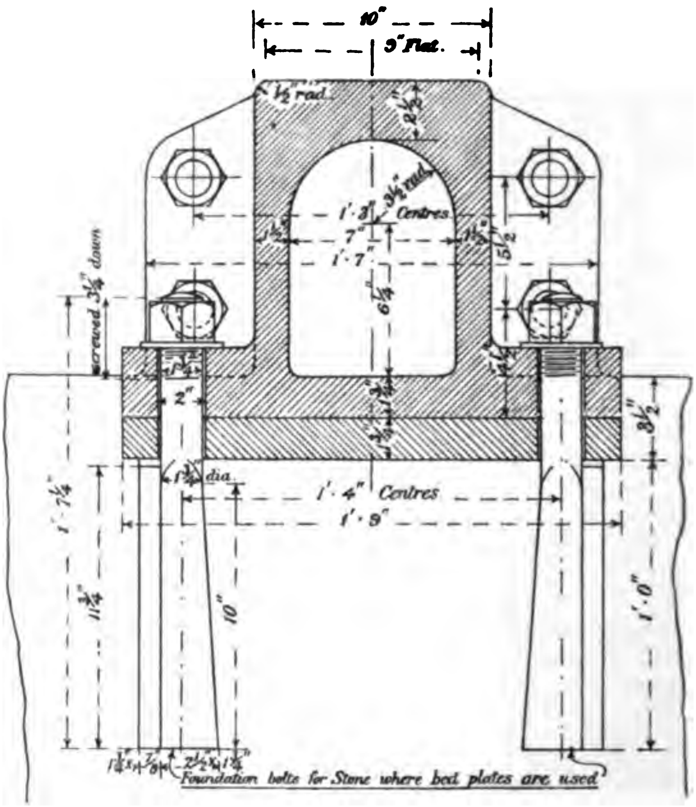
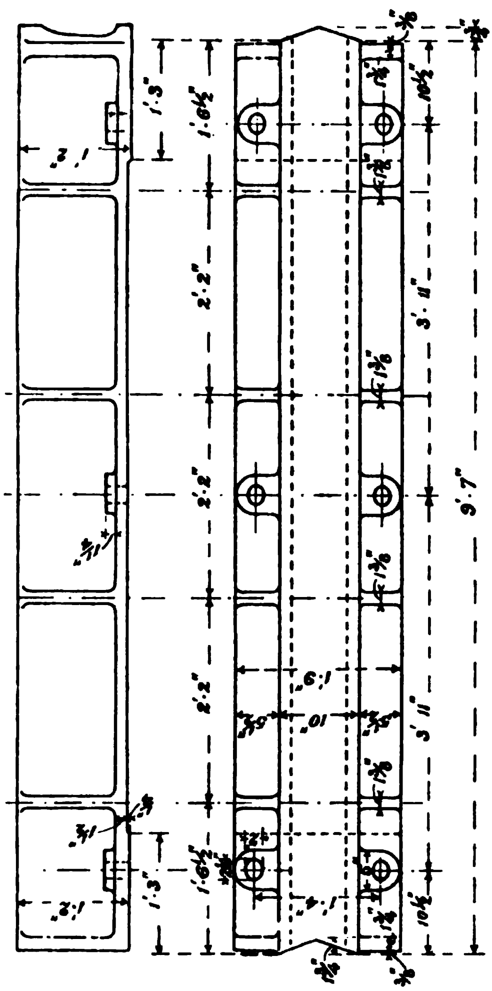


FIG. 403.
Scale 1¼ inch = 1 foot.

is 9 inches on the flat, so that the fleeting of the caisson through the allowed space of 3 inches is provided for, the edges of the roller being rounded.

The details of the roller are given in Figs. 405, 406, 407, 408, Fig. 405 being a side elevation, 406 a part front elevation and part section, 407 a plan, and 408 a sectional plan of part of the wheel casing.

The roller is 2 feet 6 inches diameter, with a total width of 6½ inches, turned true on its circumference and bored for steel spindle. The pressure of the hydraulic ram is brought upon the roller spindle by means of the casting shown, which partially



ROLLER PATH CAST STEEL.

Fig. 404.

Scale ½ inch = 1 foot.

encases the wheel, the upper surface, being shaped to a semi-spherical seating, receiving the lower end of the hydraulic ram similarly spherical shaped, as shown in Fig. 406.

The entire apparatus is provided with gun-metal guides, attached to the riveted steel framework of the caisson, for the

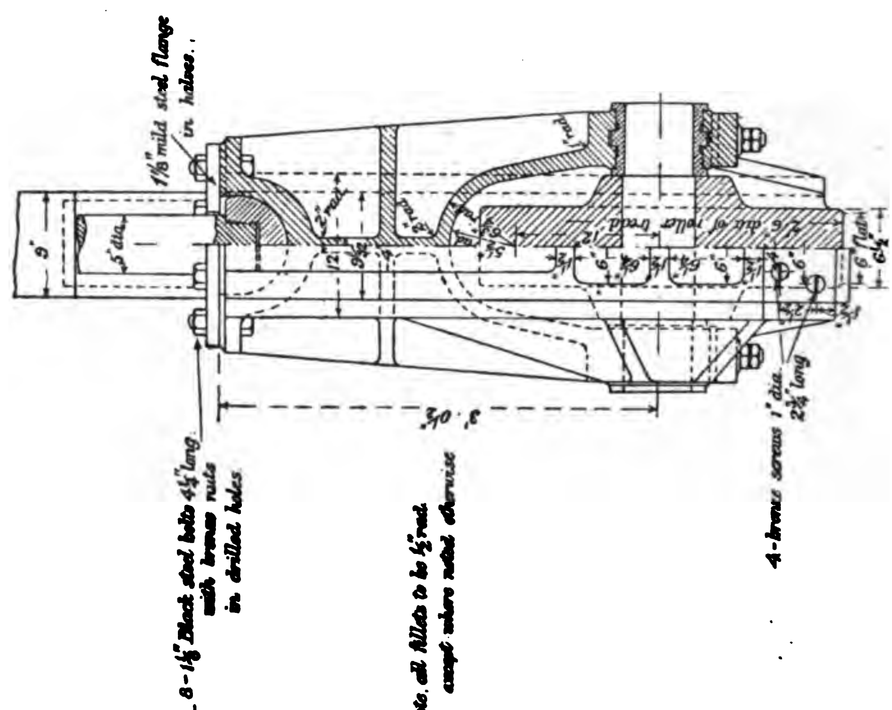


Fig. 406 (Scale ¾ inch = 1 foot).

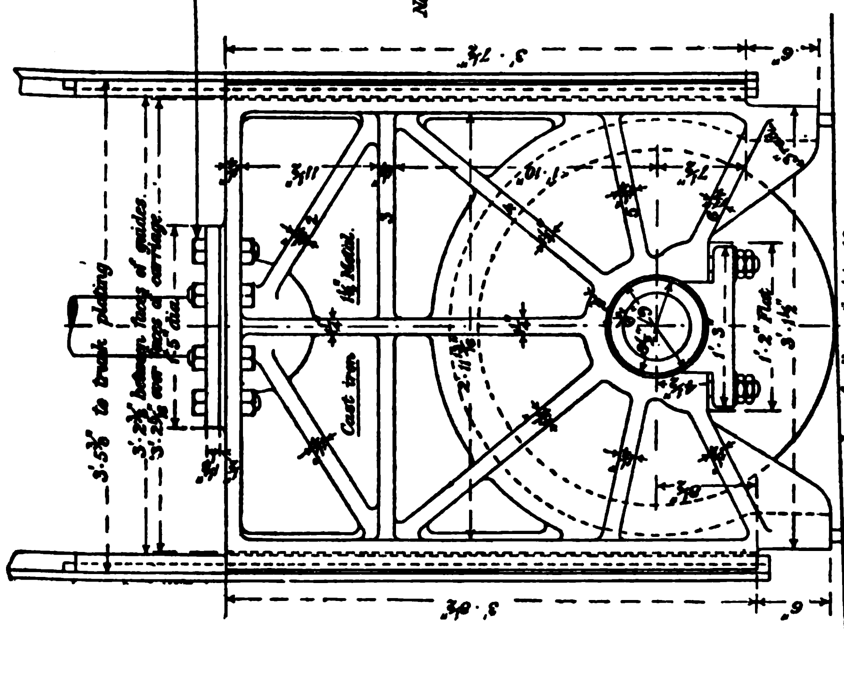


Fig. 405 (Scale ¾ inch = 1 foot).

purpose of lifting the roller as above described. India-rubber scrapers are attached to the casting fore and aft of the roller to remove mud or other obstacles from the pathway.

The hydraulic cylinders themselves are of an ordinary type,

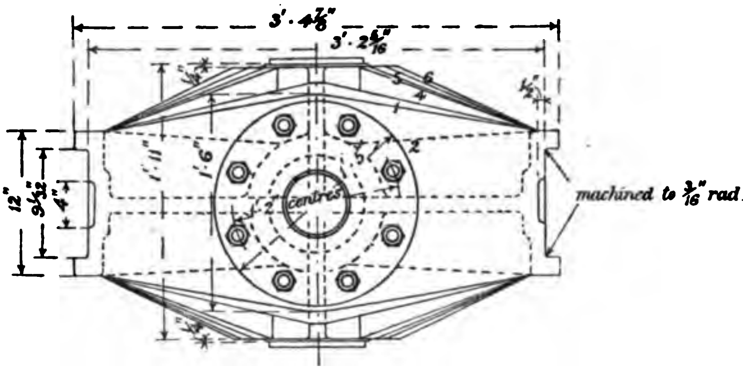


FIG. 407.
Scale ¾ inch = 1 foot

of cast steel, lined with gun metal, the net internal diameter of the cylinder being 13 inches, and the diameter of the upper portion of the piston rod 4 inches, the lower rod being 5 inches diameter.

The upper end of the rod is provided with lock nuts, capable of

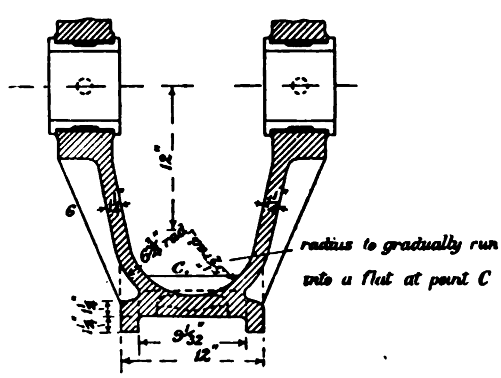


FIG. 408.
Scale ¾ inch = 1 foot.

adjustment so as to limit the amount by which the caisson keels could be lifted off their ways, a surplus amount of stroke of ram being provided to meet any possible settlement of the roller path.

Numerous tests of the cast steel used in the manufacture of the rollers and hydraulic ram cylinders will be found on p. 71 in the table of tensile and bending tests on cast steel bars.

A mud scraper is provided at both ends of the caisson on both

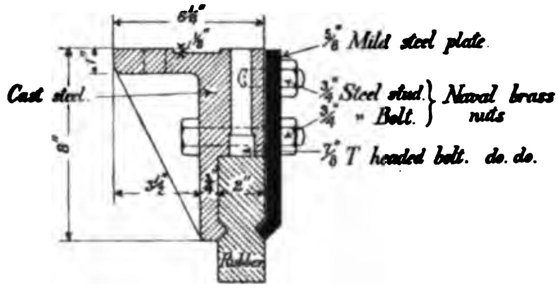


Fig. 409.
Scale 1½ inch = 1 foot.

granite keelways, placed athwart the ways. This consists of a steel casting arranged, as shown in Fig. 409, to receive a 2-inch thick slab of vulcanised india-rubber.

Handrailing to sliding caissons is usually arranged to act

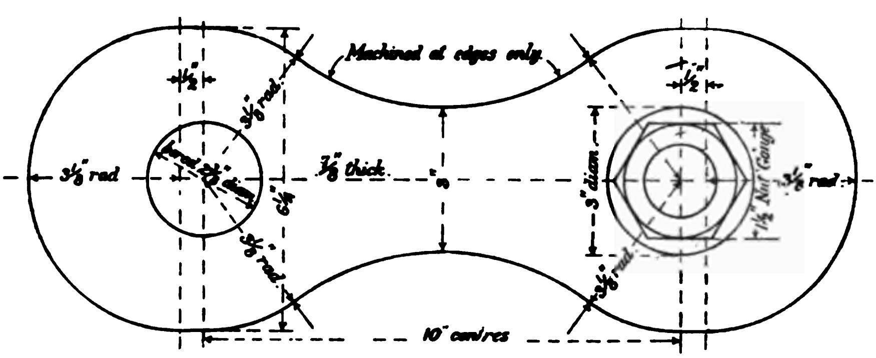


Fig. 410.
Scale 3 inches = 1 foot.

automatically, by means of lever arrangements of bars and counterbalance weights, so that upon the caisson being set in motion to enter the camber, the whole of the handrailing commences to fall into troughs prepared for the purpose, and remains stowed away until the end of the return journey, when it is caused to resume its vertical position by the power of the hauling engines.

The hauling chains of sliding caissons are sometimes of ordinary link or stud-link chain, wound on barrels and supported by rollers at intervals. A superior mechanical arrangement, though perhaps more expensive, is found in the use of the type of

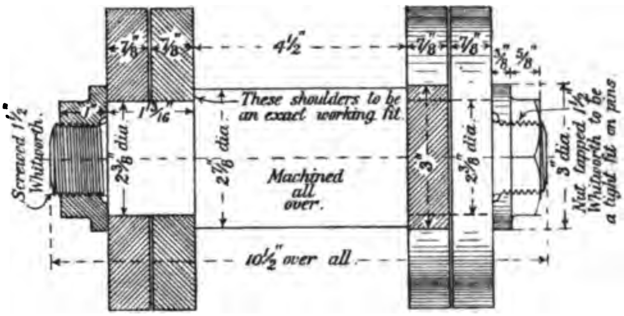


FIG. 411.
Scale 3 inches = 1 foot.

chain shown in Figs. 410 and 411, Fig. 410 being an elevation of one link, Fig. 411 a detail of pin connection, and Fig. 412 a detail of the screw coupling for the purpose of the adjustment of length, and for the taking up of preliminary stretch in the chain.

Chains of this type have been used for sliding caissons having

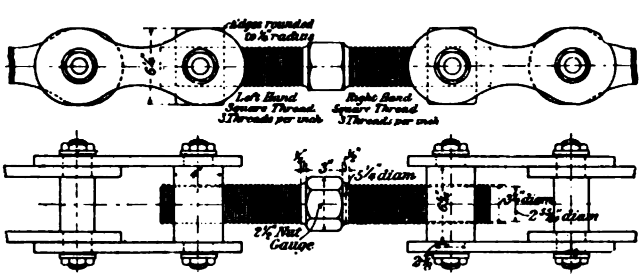


FIG. 412.
Scale $\frac{3}{4}$ inch = 1 foot.

a total mass to be hauled of 1100 tons, including ballast. Two chains are employed, one on each side, each chain consisting of two links, each link having a section of 3″ × $\frac{7}{8}$″ at the waist of the

link. The total collective sectional area of the two chains available for pull is therefore 10·5 square inches, and the chains are

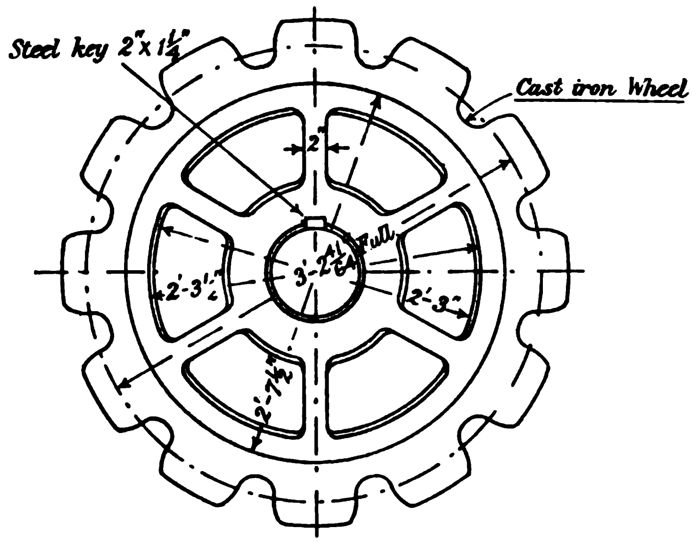


FIG. 413.
Scale ⅜ inch = 1 foot.

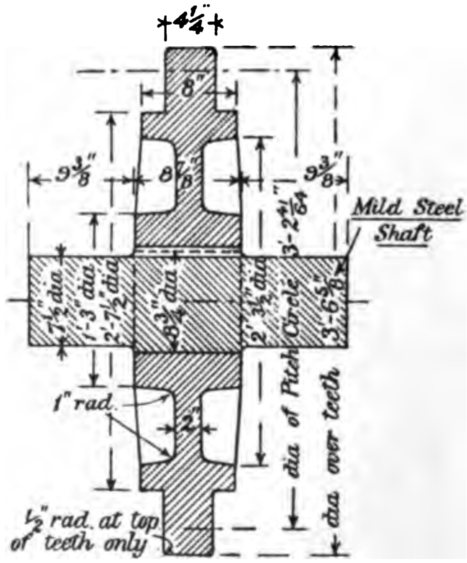


FIG. 414.
Scale ⅜ inch = 1 foot.

designed for a maximum working hauling power of 70 tons on the two chains.

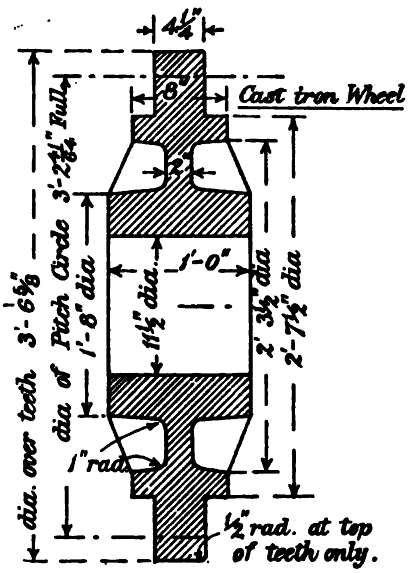


FIG. 415.
Scale ¾ inch = 1 foot.

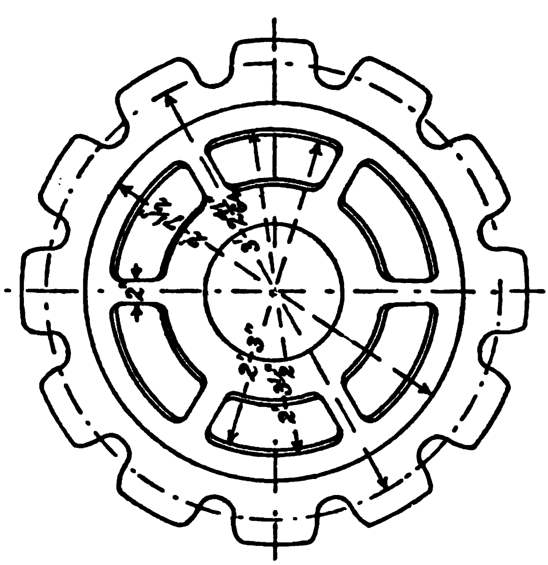


FIG. 416.
Scale ¾ inch = 1 foot.

The chains are endless, passing over two sprocket wheels, one at the forward end of the caisson camber and one at the engine end, actuated by the engine gear. The sprocket wheels of cast iron are shown in Figs. 413, 414, 415, 416, and the massive casting supporting the outer sprocket wheel and attached to the masonry of the side of the camber is shown in Figs. 417, 418.

The chains are supported for the whole of their length between the sprocket wheels by cast-iron girders, shown in Figs. 419, 420,

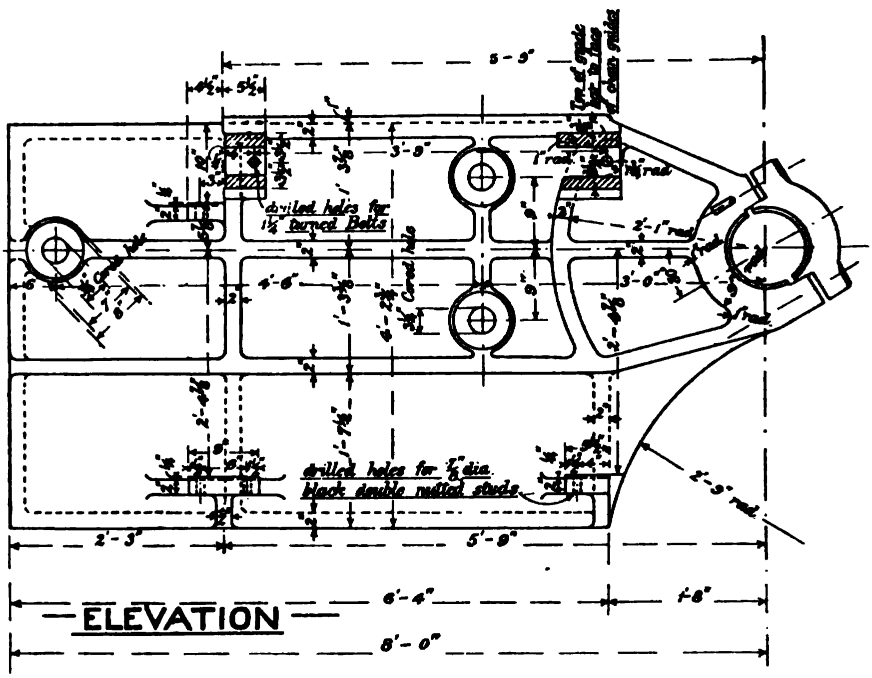
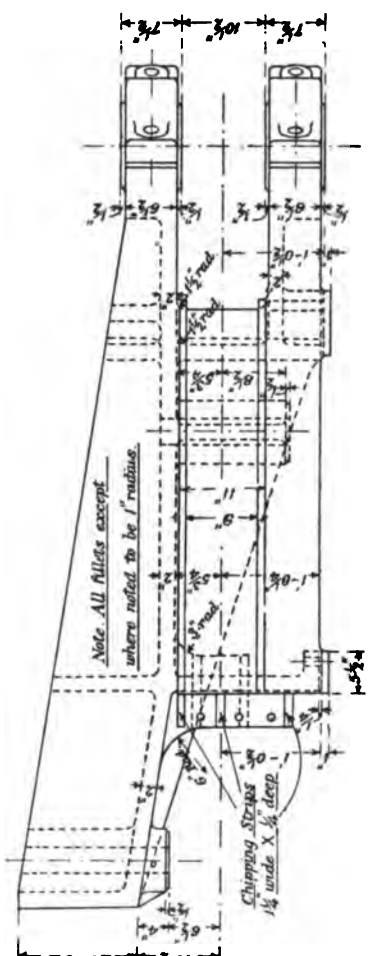


Fig. 417.
Scale ¼ inch = 1 foot.

supported by cast-iron brackets, as shown in Fig. 421, attached to the caisson camber walls, the upper surface of the girder upon which the chains travel being machined.

The chain links, pins, and nuts are of mild steel. Tests of the material employed will be found in the table of the strength of steel for special purposes, p. 46. The edges only of the links are machined, the flat sides being left with a flat surface as they come from the rolls. The holes for pins are drilled carefully to gauge,

and the pins made a close fit in the holes. The nuts are drop-forged and machined on meeting face only, made a tight fit on the screwed portion of the pin but without gripping the links, which are free to turn when revolving round with the sprocket wheels.



—PLAN—

Fig. 418.

Scale ½ inch = 1 foot.

Care is taken in shaping the teeth of the wheels to avoid any possibility of the links riding up.

The whole of the chains are each tested to a proof load of 40 tons in separate lengths of from 110 to 118 feet, the tests

being made by an hydraulic ram and the pull ascertained by a dynamometer.

Upon the application of the first pull on the chain, which

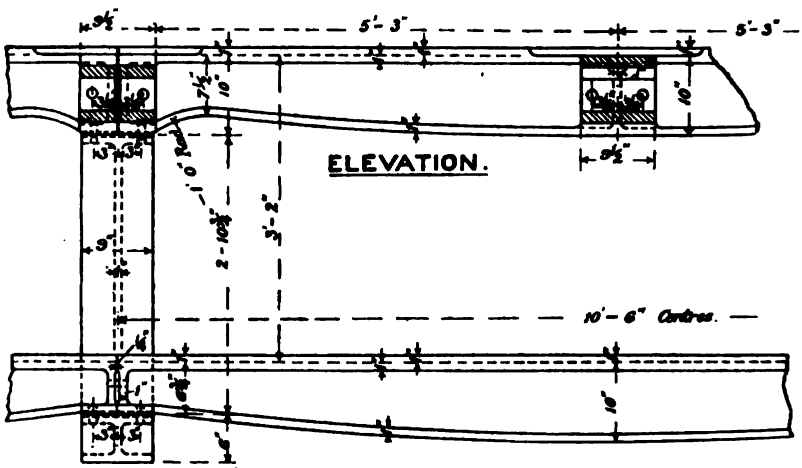


FIG. 419.
Scale ¼ inch = 1 foot.

involved tensile stresses gradually increasing from zero to 7·6 tons per square inch on the section of the waist of the link, a residuary

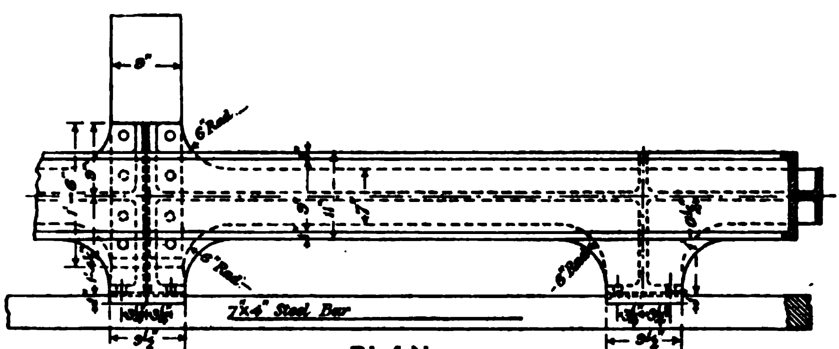


FIG. 420.
Scale ¼ inch = 1 foot.

permanent set was always found upon the removal of the load, amounting on the average to 0·0011 of the length of chain tested (about 110 to 118 feet).

When the chain was again pulled for the second and third time no further permanent set was observed, and the extensions

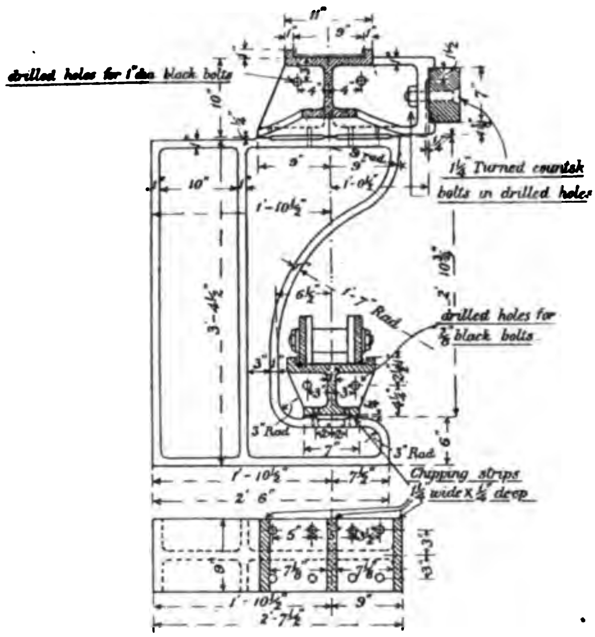


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Fig. 421.
Scale ¼ inch = 1 foot.

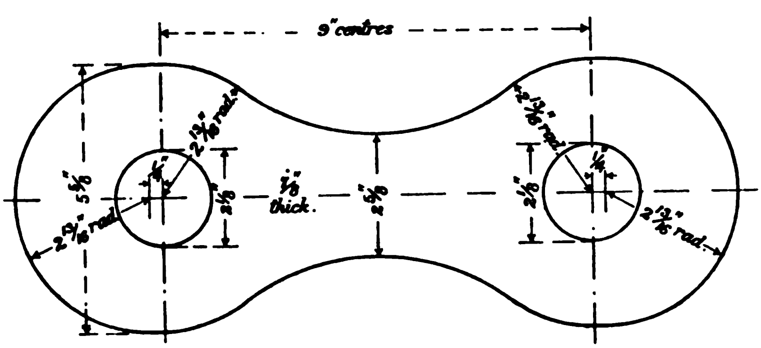


Fig 422.
Scale 8 inches = 1 foot.

of the chain were as follows, being the averages of a considerable number of observations. It will be noted that the average extensions are not exactly proportional to the increments of stress, They are, however, given as observed.

TABLE No. 39.

Observed Extensions of Hauling Chains under Tensile Stress. (10″ Links.)

Total load in tons. Second and third applications.	Tensile stress per square inch on section of the waist of the link.	Extension. (Length of chain = unity.)
5	0·952	0·00030
10	1·904	0·00057
15	2·857	0·00080
20	3·809	0·00103
25	4·762	0·00126
30	5·714	0·00137
35	6·666	0·00166
40	7·619	0·00191

The modulus of elasticity for a well-constructed chain of this type, with carefully fitted connections, and as little play between

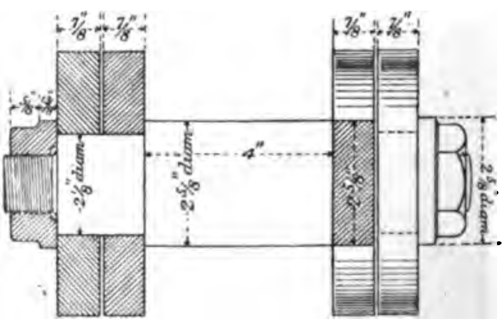


Fig. 423.
Scale 3 inches = 1 foot.

pins and holes as good workshop practice will admit of, within a range of from zero to 7·6 tons per square inch, and composed of links with a length of 10 inches centre to centre of pins, is consequently about 8,950,000 lbs. when the preliminary stretch has

been taken out of it, and when the stress on the chain is about one-fourth of the ultimate strength of the material.

A chain of similar construction, 9 inches from centre to centre of link pins, used in the hauling of caissons of smaller dimensions, and of which details of the links are given in Figs. 422, 423, tested in a manner similar to that above described, gave the following results:—

TABLE No. 40.

OBSERVED EXTENSIONS OF HAULING CHAINS UNDER TENSILE STRESS. (9″ LINKS.)

Total load in tons. Second and third applications.	Tensile stress per square inch on section of the waist of the link.	Extension. (Length of chain = unity.)
5	1·08	0·000354
10	2·17	0·000667
15	3·28	0·000900
20	4·35	0·001180
25	5·44	0·001402
30	6·53	0·001640
35	7·61	0·001948

The permanent set on the first application of the load was about 0·00102 of the length of the chain (111 to 118 feet). The modulus of elasticity derived from the second and third applications, the preliminary stretch being taken out of the chain, is about 8,750,000 lbs. for a chain of 9-inch links, centres of pins, or somewhat less than before.

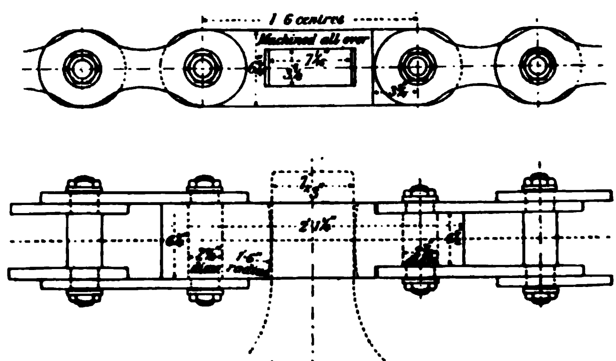
The total mass to be moved in this case was about 750 tons, including ballast, and the chain was designed for a working pull of 50 tons on the two chains.

The elasticity of long chains of this type should be carefully borne in mind in the design of all machinery details liable to be affected by the extension under load.

The attachment of the hauling chains to the bodies of the caissons is effected by means of a yoke girder. This yoke girder, which travels on wheels upon the rectangular steel bars shown in section in Fig. 421, is attached at its centre to the caisson by a massive steel hauling bar, designed to act either in tension or

compression, for pulling or thrusting, and provided with india-rubber washers to mitigate shock, while provision is made for the fleeting of the caisson from side to side. By these means the pull on the chains is equalised, the connection of the yoke girder with the chain being shown in Figs. 424, 425, a massive steel forging attached to each end of the yoke girder engaging with the hauling chain in the manner shown by dotted lines.

The following table (No. 41) gives the weight of the bodies of sliding caissons, exclusive of machinery and ballast, and inclusive of machinery but exclusive of ballast, derived from recent examples.



FIGS. 424, 425.
Scale ¾ inch = 1 foot.

The machinery referred to is only that included in the bodies of caissons themselves, principally connected with pump-work, valves, etc., and has no reference to any portion of the machinery, such as engines, hauling chains, or other gear, connected with the hauling of the caisson into or out of its position across the dock entrance.

This latter machinery, being fixed on land, has no influence on the design of the caisson as a floating body. The weights given are inclusive of all riveted steelwork and other ironwork, timber, paint, asphalte, cements, or other material.

Examples Nos. 4, 5, and 12 are provided with sluices to aid in the regulation of water level on a rising or falling tide alluded to on p. 406.

TABLE No. 41.

The Weights of Sliding Caissons.

No. of example.	Width of dock entrance at coping level.	Depth of dock entrance, coping level to sill level.	Total area of dock opening to masonry outline. Square feet.	Weight of body of caisson per square foot of area of opening exclusive of machinery and ballast. Tons.	Weight of body of caisson per square foot of area of opening including machinery, but exclusive of ballast. Tons.
1	95′ 0″	55′ 0″	4970	0·154	0·162
2	95′ 0″	55′ 0″	4970	0·154	0·162
3	95′ 0″	55′ 0″	4970	0·154	0·162
4	95′ 0″	55′ 0″	4970	0·156	0·164
5	95′ 0″	55′ 0″	4970	0·156	0·164
6	95′ 5″	49′ 0″	4462	0·118	0·122
7	95′ 0″	46′ 6″	4224	0·120	0·124
8	95′ 0″	46′ 6″	4224	0·120	0·124
9	95′ 0″	46′ 6″	4224	0·120	0·124
10	95′ 0″	43′ 6″	3974	0·124	0·131
11	95′ 0″	43′ 6″	3974	0·124	0·131
12	95′ 0″	43′ 6″	3974	0·127	0·136

CHAPTER VII.

THE PROTECTION OF STEEL SURFACES FROM CORROSION.

General remarks—The destructive effects of oxidation—Desirability of an exhaustive inquiry into the best methods of protection and the relative efficiencies of various coatings—Effects produced by locomotive gases and other agencies—Influence of these considerations upon certain types of construction—Inaccessible positions—Durability of foundation bolts, and the like, of marine structures—Attrition of shingle—Mill scale and its removal—The process of pickling mild steel—Proportions of acid bath—Tanks—Period of immersion—Subsequent processes—Coating of boiled oil—Analyses of boiled linseed oil—First coat of paint—Oxide of iron paint—Analyses of oxide of iron paint—Analyses of lead colour or grey paints.

THE destructive effects of oxidation upon the surfaces of metallic structures must necessarily engage the most careful consideration of the designer, and as our experience of the life-history of such structures increases and enlarges with the lapse of time, it becomes increasingly evident that the neglect of such precautions as are in our power must have a serious effect in shortening the period of time within which a metallic structure, such as a steel girder or roof principal, can maintain its working efficiency.

The powerful influences of oxidation in structural iron or steel work are everywhere in evidence, while the relative protecting efficiency of the various preservative coatings used can only be determined by the lapse of time, and a careful comparison of the conditions under which they are used; and notwithstanding the number of coatings which are in the market, an exhaustive inquiry into their relative efficiency and the best possible means of protecting steelwork is still required.

The rapidity with which oxidation can be set up in any structure varies so much with circumstances that it is impossible to give any general rule or estimate by which to calculate its probable effects.

As an example, we have no difficulty in observing the very destructive effects produced by the gases issuing from a locomotive funnel. Such structures as the lower member of the wind screen at the end of a railway station, placed immediately over the position usually occupied by the engine when standing, and consisting, it may be, of angles and lattice bars of light scantling, have been wasted to destruction, while the webs of plate girders in similar positions have had large holes eaten right through in a few years.

The webs of plate girders in promenade pier construction at the point of junction of the girder with the pier decking have also been known to be eaten through into holes, and the plane of junction of the metallic structure of highway road bridges with the surface of the road or footways appears also peculiarly liable to the destructive attacks of oxidation or electrolysis, combined with the chemical effect of the impurities of roadway drainage.

It is, unfortunately, frequently the case that the parts most affected are precisely those least able to be reached by the painter's brush, and least liable to be detected, and this consideration has some force with respect to various details of construction which are, from every other point of view, desirable and efficient. For example, it is frequently found convenient to frame the upper and compressive member of a roof truss with two angles in place of one tee, or similarly the bracing members, or even the tie bars, when they are flats or eye bar links. Such construction is attractive in several ways. Connecting plates are easily inserted, rivets are in double shear, and the whole detail works out neatly. But it is to be feared that when the time comes round (often too long delayed) for repainting, those surfaces which are too close together to be conveniently reached by the paint brush will be neglected, unless special precautions be taken to ensure that they receive their fair share of the protective coating.

A similar example on a larger scale may be found in the tension diagonals of large lattice girders, frequently formed of two separate flat bars placed close together, or nearly so, but not so close as to prevent the entrance of moisture, which gradually trickling down, and perhaps also led by capillary attraction, sets up a powerful oxidising action between the surfaces, to such an extent as in course of time to swell the bars by an excessive accumulation of rust out of their true line, and, while reducing their cross-section, also adds to the stresses in action in the bar.

An instance of this is to be found in a well-known metropolitan bridge, where it has been found necessary to tack the bars together by rivets or bolts, and otherwise to render the surfaces watertight.

It would appear desirable in such cases so to tack the separate bars together at the edges as to render the interior surfaces as far as possible impervious to moisture, while the effective sectional area in tension need not be reduced to any greater extent than would ordinarily occur in the riveted connection at the ends of the bar.

The interior surfaces of riveted box girders and of certain sections of riveted columns are obvious examples of construction where the paint brush can never reach, unless the closed cell be of sufficient dimensions to admit of the working inside of man or boy. This can be realized in the design of large box girders, say, of 60 feet and upwards, and in such instances covered manholes are sometimes provided for access.

It is so far satisfactory to find that in small box girders, inaccessible inside, where the ends are practically hermetically sealed and the entrance of moisture or impure air prevented, experience appears to show that few signs of deterioration are to be found after the lapse of some years.

The imbedding of ironwork in Portland cement or concrete also tends to preservation, although much depends upon the amount of close contact actually obtained and the adhesion between the surfaces.

The durability of foundation bolts, rag bolts, and the like depends largely upon this, and there can be no doubt that the condition of much iron and steel work in such situations is greatly a matter of speculation.

In all probability in respect of the great mass of iron and steel structures now existing, a future generation of engineers and architects will not lack employment in their renovation or repair.

The durability of iron and steel work in certain marine structures is frequently very powerfully influenced not only by corrosion, but also by the erosive action of sand or shingle. The cast-iron piles of promenade piers, where exposed to the action of shingle set in motion by tidal or wave action, are sometimes found to be seriously affected thereby, the scouring nature of the shingle or sand producing effects even more serious than oxidation. The same may be said of outfall sewer pipes and the like. Probably the best protection in these cases is a substantial coating of Portland

cement mortar or concrete. Roman cement is also used for the same purpose.

The protection of steel surfaces commences, or at all events in good work ought to commence, at a very early period in the construction.

Plates and bars are, as a rule, delivered in the girder or roof builder's yard in the condition in which they come from the rolling mills, that is to say, with no protective medium on their surfaces, unless the thin greyish-black scale, called mill-scale, which is formed during the processes of rolling, and is loosely attached to their surfaces, can be so called.

This scale, which occurs in thin flakes, is but slenderly attached, and can be more or less easily removed. As a consequence, paint applied to such a surface is not unlikely to come off with the scale, leaving bare places unprotected and exposed to oxidation, although in work where the process of "pickling," now to be described, is not used, the paint and the scale probably get mixed up to a certain extent together, unless the surfaces are carefully scraped before painting, which should be the case when pickling is not employed, or if the process of cleaning by means of the sand blast is not adopted.

Where the class of work requires the complete removal of this scale the process of "pickling" is frequently adopted, and it is possibly the absence of this precaution which has, in some cases, led to complaints as to the premature oxidation of mild steel.

The process consists in the immersion of the steel plate or bar in a bath of diluted acid, commonly specified to consist of one part of hydrochloric acid to nineteen parts of water. This liquid is contained in timber tanks of substantial construction, and of dimensions suitable to the largest plate or longest bar to be dealt with. Plates should not be laid flat upon one another when "pickled," but retained in a vertical position, with sufficient space between each. For this purpose tanks for plates are frequently fitted with racks at the ends, into which the plates are dropped, and thus held upright, the handling of the plates being also facilitated.

Bars are laid horizontally and loosely together. The action of the dilute acid upon the surface of the plates or bars is accompanied by the ebullition of a considerable amount of gas, which rises to the surface of the liquid in numerous bubbles, large and small, and this ebullition is a measure of the proper activity of the bath.

After an immersion which may average from seven to fourteen hours, according to the freshness or activity of the bath, and

governed to some extent by the time of year, the bath being somewhat more active in summer time, the plates or bars are hoisted out of the bath, and thoroughly washed with pure water by means of a hose. They should then be stacked until they are required for the further shop processes of marking off, punching or drilling, shearing, etc.

The student will observe, in passing, the effect produced upon wrought-iron chains which have been immersed in the acid for a considerable time, in laying bare and bringing to view the fibrous nature of this material.

The surfaces of the plates or bars, when properly pickled, after removal from the bath, will be found clean and smooth to the touch, and free from all appearance of scale, but will soon take on the appearance of premature oxidation, with an evenly distributed thin light yellow rust when exposed to the weather and not further protected.

In order to prevent this, and for the protection of the material during the period, often of considerable length, occupied in the various shop processes of conversion into riveted work, and during which time the plates or bars are often exposed in the contractor's yard to all the vicissitudes of weather, it is frequently specified that the whole of the steelwork shall be covered with a coating of boiled linseed oil, which is brushed on, and if of proper quality will be found in about twenty-four hours to have dried to a tough film, forming an effective preliminary and temporary protection until such time as the first coat of paint can be applied. Manufacturers will occasionally object to this process on the grounds of the slippery nature of the coating making the handling of the plates, etc., somewhat difficult; but it may be argued that the advantages counterbalance the objection.

The following analyses of boiled linseed oil, made as samples of the oil actually used in extensive contracts and used for protecting steelwork in the manner above described, will be found of interest as showing what chemical composition should be looked for, and the variations which will occur in ordinary practice.

Chemical analyses of samples of boiled linseed oil—

(1)	Specific gravity at 60° Fahr.	0·947
	Mineral acid	Nil
	Unsaponifiable matter	1·390 per cent.
	Ash	0·060 ,,

This sample dries well when exposed on glass.

(2) Specific gravity at 60° Fahr.	0·946
Mineral acid	Nil
Unsaponifiable matter	1·20 per cent.
Ash	0·30 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... „ „ 16 „

(3) Specific gravity at 60° Fahr.	0·942
Mineral acid	Nil
Unsaponifiable matter	1·18 per cent.
Ash	0·22 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... „ „ 16 „

(4) Specific gravity at 60° Fahr.	0·946
Mineral acid	Nil
Unsaponifiable matter	1·90 per cent.
Ash	2·22 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a fairly hard tough film in 24 hours.
2. At 100° Fahr. ... Dried to a hard tough film in 16 hours.

This analysis shows the sample to contain a good deal of "foots," but it is otherwise of satisfactory quality.

(5) Specific gravity at 60° Fahr.	0·943
Mineral acid	Nil
Unsaponifiable matter	0·42 per cent.
Ash	0·02 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... „ „ 16 „

The above figures show this sample to be of satisfactory quality.

(6) Specific gravity at 60° Fahr.	0·962
Mineral acid	Nil
Unsaponifiable matter	1·19 per cent.
Ash	2·70 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a somewhat sticky but fairly tough hard film in 24 hours.
2. At 100° Fahr. ... Dried to a hard tough film in 16 hours.

In this sample the specific gravity and the percentage of ash are unduly high, and the drying of the oil not wholly satisfactory.

(7)	Specific gravity at 60° Fahr.	0·946
	Mineral acid	Nil
	Unsaponifiable matter	4·28 per cent.
	Ash	0·20 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... ... Sticky after 24 hours.
2. At 100° Fahr. ... ... Somewhat sticky after 16 hours.

The analysis shows this sample to be adulterated and to contain a large amount of "foots."

(8)	Specific gravity at 60° Fahr.	0·949
	Mineral acid	Nil
	Unsaponifiable matter	4·88 per cent.
	Ash	0·08 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Failed to dry in 24 hours, very sticky film.
2. At 100° Fahr. ... Dried to a fairly tough but slightly sticky film in 16 hours.

This analysis shows the sample to be also adulterated.

(9)	Specific gravity at 60° Fahr.	0·946
	Mineral acid	Nil
	Unsaponifiable matter	0·96 per cent.
	Ash	0·24 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... „ „ 16 „

The above analysis shows the sample to be of satisfactory quality.

(10)	Specific gravity at 60° Fahr.	0·945
	Mineral acid	Nil
	Unsaponifiable matter	1·09 per cent.
	Ash	0·27 „

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... " " 16 "

(11)	Specific gravity at 60° Fahr.	0·946
	Mineral acid	Nil
	Unsaponifiable matter	1·05 per cent.
	Ash	0·01 "

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... " " 16 "

(12)	Specific gravity at 60° Fahr.	0·947
	Mineral acid	Nil
	Unsaponifiable matter	1·17 per cent.
	Ash	0·25 "

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... " " 16 "

This sample of satisfactory quality.

(13)	Specific gravity at 60° Fahr.	0·943
	Mineral acid	Nil
	Unsaponifiable matter	1·09 per cent.
	Ash	0·07 "

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... " " 16 "

These results show the sample to be of a satisfactory quality.

(14)	Specific gravity at 60° Fahr.	0·950
	Mineral acid	Nil.
	Unsaponifiable matter	1·28 per cent.
	Ash	0·22 "

When exposed on glass in a thin film—

1. At 60° Fahr. ... Dried to a hard tough film in 24 hours.
2. At 100° Fahr. ... " " 16 "

The preliminary processes of construction having been completed, and the various plates and bars having gone through the operations of shearing, straightening, planing, punching or drilling, and the like, the assemblage of the parts in the contractor's yard and putting together with service bolts commences, and at as early a stage as is possible the temporary coating of boiled oil should be superseded by the first coat of paint. In some specifications it is laid down that the oil coat should be carefully scraped off before the first coat of paint is applied. It is doubtful to what extent such a stipulation is usually complied with, nor does it appear to be of great moment.

The first coat of paint has usually to do duty as a protective medium for a considerable period of frequently indefinite duration. Some time may elapse, for example, before the material temporarily erected in the contractor's yard is finally put together on the site, and in the interim much exposure to weather may take place, transhipment by sea and the accompanying possibilities of damage by salt water may ensue, while a good deal of rough usage in the various operations of loading up and unloading may be expected. Against all these contingencies the first coat is a valuable preservative.

Among the various and numerous paints recommended or used for the protection of steelwork, none is perhaps more generally considered as one of the best, or at all events more frequently employed, than the well-known oxide of iron paint.

Other paints, for more or less decorative purposes, are not here under consideration, the practical question before us being simply protection from corrosion and decay.

This paint gives rise to the familiar reddish-brown or chocolate colour which characterizes the appearance of much structural steel and iron work, although there are frequently cases in which, for the purpose of maintaining as uniform a temperature as possible, and to avoid absorption of the sun's rays, a white, stone colour, or light grey, paint may be used.

The following chemical analyses of oxide of iron paints, made as samples of the paints used in extensive contracts for constructional and other classes of steelwork, will be found instructive as to their composition, while the variations are such as may be expected to arise in practice in the use of paint of good quality.

Analyses of red oxide of iron paint—

(1)

	Per cent.
Oil	41·23
Insoluble siliceous matter	4·72
Ferric oxide (Fe_2O_3)	45·31
Calcium carbonate ($CaCO_3$)	4·39
Magnesia (MgO)	0·08
Sulphuric anhydride (SO_3)...	1·67
Combined water, alkalies, and loss ...	2·60
	100·00

(2)

	Per cent.
Oil	16·86
Insoluble siliceous matter	8·05
Ferric oxide and alumina (Fe_2O_3 and Al_2O_3)	65·41
Calcium carbonate ($CaCO_3$)	5·10
Magnesia (MgO)	0·76
Combined water and loss	3·82
	100·00

(3)

	Per cent.
Oil	14·66
Insoluble siliceous matter	7·90
Ferric oxide	68·07
Lime	2·94
Magnesia	0·49
Sulphuric anhydride	0·78
Combined water and loss	5·16
	100·00

(4)

	Per cent.
Oil	15·18
Insoluble siliceous matter	6·67
Ferric oxide	71·10 [1]
Lime...	2·26
Magnesia	0·60
Sulphuric anhydride	0·25
Combined water and loss	3·94
	100·00

[1] Corresponding with 83·82 per cent. of ferric oxide calculated on the dry pigment.

(5)

	Per cent.
Oil	14·98
Insoluble siliceous matter	6·69
Barium sulphate	1·73
Ferric oxide	70·40
Calcium carbonate	3·79
Magnesia	0·34
Sulphuric anhydride	0·09
Combined water and loss	1·98
	100·00

The following are analyses of lead colour or grey paints used for constructional steelwork and for machinery:—

1. Analysis of a sample of lead colour paint—

	Per cent.
Oil	23·83
Basic lead carbonate	72·42
Carbon	0·08
Insoluble residue ($BaSO_4$)	2·74
Calcium carbonate ($CaCO_3$)	0·93
	100·00

This sample accepted.

2. Analysis of a sample of lead colour paint—

	Per cent.
Oil	29·85
Barium sulphate	2·37
Basic lead carbonate	62·21
Calcium carbonate	4·51
Carbon	0·60
	99·54

This sample accepted.

3. Analysis of a sample of lead colour paint—

	Per cent.
Oil	9·61
Barium sulphate	42·12
Basic lead carbonate	34·65
Calcium carbonate	11·40
Ferric oxide and alumina	0·27
Magnesia	0·14
Carbon	1·50
Combined water, alkali, and loss	0·31
	100·00

This sample was rejected as of inferior quality.

4. Analysis of a sample of lead colour paint used for machinery—

	Per cent.
Oil	27·64
Basic lead carbonate	70·03
Insoluble siliceous matter...	0·87
Ferric oxide	1·03
Lime	0·26
Magnesia	0·13
	99·96

This sample accepted.

5. Analysis of a sample of lead colour paint—

	Per cent.
Oil	26·46
Basic lead carbonate	65·97
Carbon	0·48
Insoluble residue ($BaSO_4$)...	5·15
Calcium carbonate	1·71
Magnesia	0·19
	99·96

This sample considered to be of satisfactory quality. Accepted.

6. Analysis of a sample of lead colour paint—

	Per cent.
Oil	10·24
Insoluble matter (barium sulphate) ...	5·46
Basic lead carbonate	73·89
Carbon	1·28
Ferric oxide (alumina)	0·32
Calcium carbonate	8·04
Magnesia	Trace
Combined water, alkali, and loss	0·77
	100·00

This sample considered to be of satisfactory quality. Accepted.

7. Analysis of grey lead paint—

	Per cent.
Oil	23·53
Turpentine	2·54
Pigment	73·93
	100·00

Analysis of the pigment—

	Per cent.
Basic white lead	86·97
Insoluble siliceous matter...	8·86
Ferric oxide	0·16
Calcium carbonate	0·66
Carbon	3·04
	99·69

This sample was rejected.

Analysis of a further sample—

	Per cent.
Oil	23·45
Turpentine	3·50
Pigment	73·05
	100·00

Analysis of the pigment—

	Per cent.
White lead	95·54
Carbon	1·72
Insoluble siliceous matter	1·58
Ferric oxide and alumina	0·46
Lime	Nil.
	99·30

This sample was accepted.

For the preservation of steel surfaces exposed to corrosion by prolonged immersion in sea water, especially in positions difficult of access, various kinds of cements have been used. The composition of the cement referred to in Chap. VI., p. 402, and used as a coating in the bilges of caissons and other similar situations, is as follows :—

Mineral pitch	250 lbs.
Mineral tar	6 gallons
Roman cement	360 lbs.
Lime, fine white	56 „
Resin, fine black	14 „
Oil of naphtha, or black naphtha ...	1 gallon

This composition gave an approximate weight of 102 lbs. per cubic foot, and formed a hard tenacious coating with an excellent surface, when applied to steel plates.

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THE END.

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